The Association

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Engineering and Shipbuilding Draughtsmen.

The Design of Structural Welded Details.

By FRANK H. ABRAHAMS, A.M.I.Struct.E.

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SYNOPSIS.

Side and End Fillet Welds—Table of Strength of ditto—Intermittent Fillets—Tee Fillets—Diagonal Fillets—Square Butt Welds—V-Butt Welds—U-Butt and J-Butt Welds—Strength of Butt Welds—Horizontal and Vertical Fillets—Vertical Welding—Overhead Welding—Inclined Welding—Distortion—Connection of Angles—Beam Seatings on Stanchions—Eccentric Beam Seatings—Non-Rigid Beam Connections—Rigid Beam Connections—Stanchion Bases subject to Vertical Load only—Stanchion Bases subject to Bending Moment—Stanchion Crane Caps subject to Bending Moment—Stanchion Splices—Design of Beams and Plate Girders—Plate Girder Stiffeners—Procedure for welding Plate Girders—Crane Brackets on Stanchion Flanges—Roof Truss Details—Lattice Girder Details—Staircase Details—Standardising of Welding Technique—Multiple Run Welds—Exercises—Bibliography.

INTRODUCTION.

IN changing over from riveted to welded construction, it should not be assumed that all that is required is to maintain the same type of connection, and to replace the rivets by welds of equal strength. An entirely different technique is called for in welded design. For example, the end connection of a beam is usually made with cleats in riveted work, but the cleats may be omitted completely in a welded detail. Similarly in a plate girder, flange angles are required in riveted work to connect the web plate to the flange plates, but in welded work the angles may be omitted. A much better understanding of the requirements for welded work will be obtained if it is assumed that the welded construction is an alternative to a steel casting rather than to a built-up riveted design. The draughtsman is, in effect, building up the equivalent of a casting by introducing welds in place of the fillets in a casting.

British rolled sections were designed for riveted work, and are often unsuitable and uneconomical for welded construction. Where this is so, the designer must either make the best use possible of them, or alternatively build up his own sections from plates welded together. The latter method is usually the more economical. For example, in riveted work a stanchion shaft is usually a joist with

or without flange plates, but it may be more economical in welded work to build up an H-section from flats and plates, varying the thickness and width to suit the load at different heights.

It will, therefore, be found that details for welding show considerable simplification, and are more economical in steel, as compared with riveted work, as the sections can more easily be changed to suit load variations, and joint details often consist solely of a weld without added joint materials. This is an important point to bear in mind in assessing the relative costs of riveted and welded work, as although the actual operation of making a welded connection may be more expensive than that of drilling and riveting (but not always so), there is considerable saving of material and consequently of the labour that would be employed in fabricating the material thus saved. This does not mean necessarily that a welded connection is cheaper than a riveted connection, but it does mean that where considerable simplification can be obtained by welding, a case has been established for examining relative costs.

Also, in certain cases, conditions may make it very difficult to include a satisfactory riveted joint, whereas a welded joint may be comparatively simple. In such cases, the welded construction is to be preferred, as simplification of details is always desirable from the point of view of good construction.

It has been assumed that the reader already possesses a reasonable knowledge of elementary Theory of Structures, and attention has been concentrated on applying standard design procedure to the particular problem of welded details. The majority of the calculations are, however, of a simple nature and should be easily followed.

In so far as the application of welding to structural steelwork detailing is still to some extent in the experimental stage; the calculations given here should be taken as approximate, erring always on the safe side and making suitably safe assumptions.

It will be appreciated that it is not possible in a pamphlet of this size to deal in more than general terms with problems of fabrication, many of which can, in fact, only be solved by the method of trial and error. The reader is referred for further information on this point to the "Report on the Treatment of Welded Structures," issued by The Institution of Structural Engineers, which gives very full information regarding samples and experiments, particularly in relation to the most difficult of all welding problems, that of distortion.

Fig. 1 shows typical examples of side and end fillet welds. The basis of weld strength is the thickness of the weld at the throat, so that the effective area of a fillet weld, for design purposes, is the throat thickness, multiplied by the effective length. The strength of a weld can most conveniently be stated as so many tons per

inch run. This is the usual procedure of design and the method generally adopted in tables of strengths of welds. It will be obvious that if too much weld metal is deposited and the shape of the weld is too convex, the connection is wasteful, while if too little metal is deposited and the shape is too concave, the connection will be too weak. The correct profile should leave the weld slightly convex, as shown in the enlarged detail, and the length of run which can be obtained from a standard electrode is based on this profile. The short returns to the weld are not included in strength calcula-

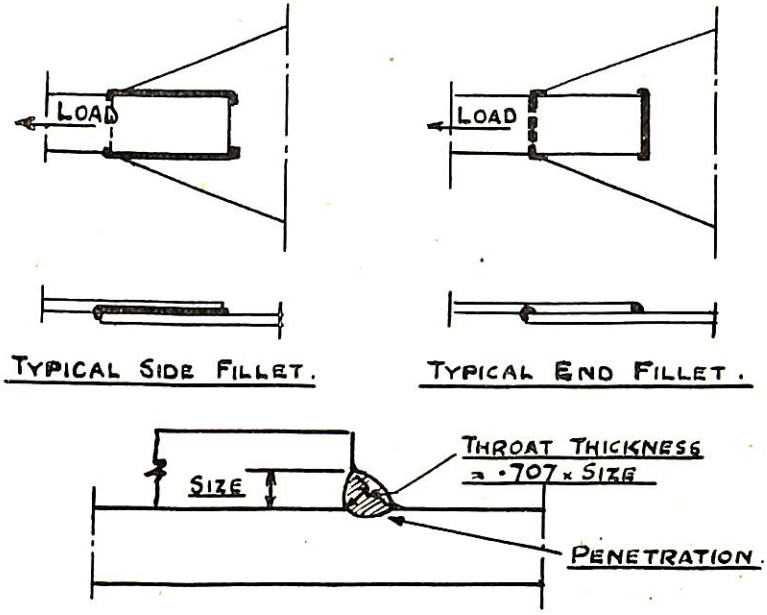

Fig. 1.

tions, but are necessary to ensure that the full length of the weld run is effective. This is due to the fact that a crater is usually formed at the beginning and end of a run, and it is doubtful if these portions are effective in taking load. Where it is not possible to include a return, the effective length for design purposes is assumed to be the specified length, less a length equal to twice the weld size. Thus a $\frac{1}{4}$″ fillet, 6″ long, without returns, would be assumed to have an effective length of $5\frac{1}{2}$″.

L.C.C. Regulations issued in December, 1937, specify a permissible stress of 6 tons per square inch for end fillet welds, and 5 tons per square inch for side fillet welds. Table 1 gives the strength per lineal inch of side and end fillets in accordance with these regulations.

When side fillets are used for the end connection of a member, the effective length of the welds should be at least equal to the distance between them, *i.e.*, the width of the member. This requirement corresponds to the requirement in riveted work to make the length of a batten plate at least equal to the width. In practice, however, this requirement is often ignored.

TABLE 1.

Strength in Tons per Lineal Inch of Fillet Welds.

(L.C.C. Regulations).

Size of Fillet.	Throat Thickness.	End Fillets at 6 Tons per Sq. In.	Side Fillets at 5 Tons per Sq. In.
$\frac{1}{8}$″	0·088″	0·53	0·44
$\frac{3}{16}$″	0·132″	0·79	0·66
$\frac{1}{4}$″	0·177″	1·06	0·89
$\frac{3}{8}$″	0·265″	1·59	1·33
$\frac{1}{2}$″	0·354″	2·12	1·77
$\frac{5}{8}$″	0·442″	2·65	2·21
$\frac{3}{4}$″	0·530″	3·18	2·65
$\frac{7}{8}$″	0·619″	3·71	3·10
1″	0·707″	4·24	3·54

A single end fillet should not be used for the end connection of a member. Either two end fillets should be used, or if this is not possible, two side fillets should be introduced in addition to the end fillet.

Where it is not possible or convenient to take up all the load in side and end fillets, slots or holes may be introduced to increase the length of weld metal, as shown in Fig. 2. It is important to note in such cases that the weld metal must not fill the slot or hole, or the effect of the procedure is partly nullified. This means that the slot or hole must be of sufficient width to enable the weld metal to be deposited round the perimeter without difficulty. A further point to be noted, when slots or holes are used, is that there must be sufficient length of weld in advance of the slot or hole to transmit

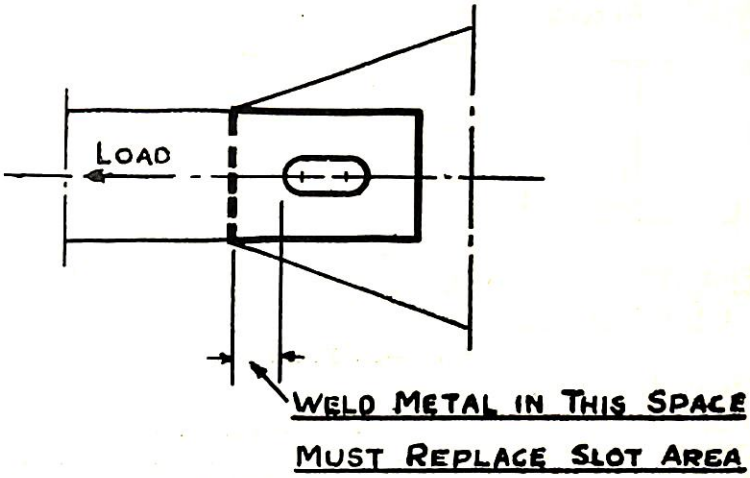

Fig. 2.—Typical Example of Slot.

load equivalent to the sectional area lost at the slot, in order to ensure that the parent metal is not weakened. For example, if the slot reduces the sectional area by 20%, then at least 20% of the load must have been transferred through the welds in advance of the slot. This procedure is similar to that for designing rivets in advance. Slot welds should not, however, be used at all if they can be avoided, as they are difficult to lay.

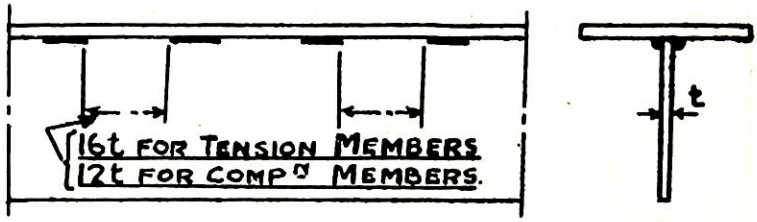

Fig. 3.—Intermittent Fillet Welds.

Intermittent Fillets.

Where continuous welds are not necessary for strength, intermittent welding may be adopted, providing a continuous fillet is not necessary for weathering or other similar purpose. For tension members, the gap between the welds should not exceed 16 times the thickness of the thinner plate, and for compression members 12 times the thickness of the thinner plate (see Fig. 3). This requirement corresponds to the maximum pitching of rivets in riveted members.

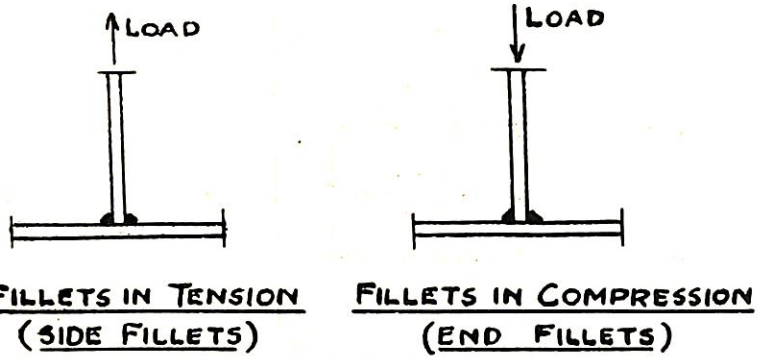

Fig. 4.—Tee Fillets.

Tee Fillets (Fig. 4).

Where the welds in a tee-fillet are in tension, they should be designed as side fillets at 5 tons per square inch, but where they are in compression they may be designed as end fillets at 6 tons per square inch.

Diagonal Fillets (Fig. 5).

These are obviously neither side fillets nor end fillets, so that their actual working stress is somewhere between 5 and 6 tons per square inch. Referring to Fig. 5, if the angle of slope is 0°, the fillet becomes a side fillet with a working stress of 5 tons, while if the angle of slope is 90°, the fillet becomes an end fillet with a working stress of 6 tons, so that the actual allowable stress should vary between 5 and 6 tons as the angle of slope varies. This point

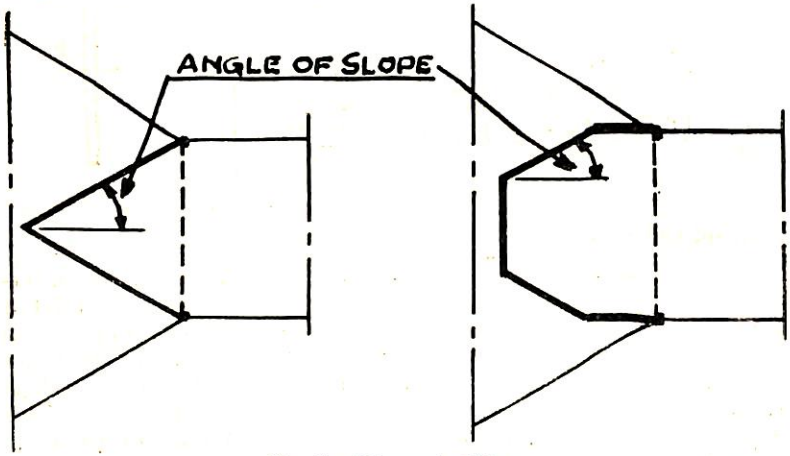

Fig. 5.—Diagonal Fillets.

is not usually of great importance, although it is worth while bearing in mind, but L.C.C. Regulations specify that the working stress should be taken at 5 tons per square inch.

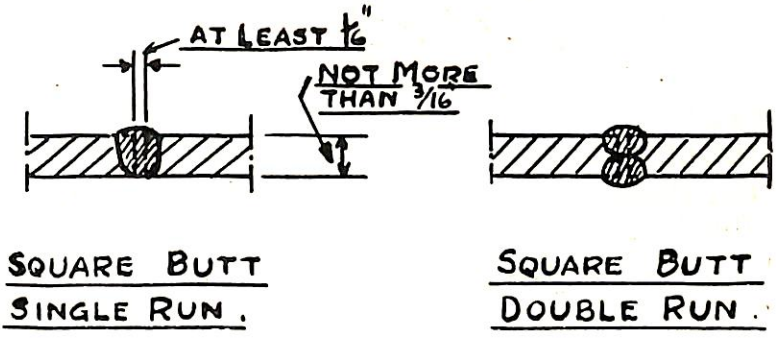

Fig. 6.

Square Butt Welds (Fig. 6).

It is possible to make quite good square butt welds for plates, up to $\frac{3}{8}$" thick, but L.C.C. Regulations specify a maximum thickness of $\frac{3}{16}$" for these welds, with a gap of not less than $\frac{1}{16}$" between the plates. The objection to a square butt for plates over $\frac{3}{16}$" is the tendency of the weld metal as it cools and contracts to decrease the gap between the plates as the welding advances, thereby decreasing the effective penetration. If square butt welds are used for plates over $\frac{3}{16}$", tack welds should be introduced at intervals to hold the plates in correct position and to maintain the gap.

V-Butt Welds (Fig. 7).

For plates over $\frac{3}{16}$" thick, single or double V-butt welds are used for general structural work. Double V-butts are used for plates over $\frac{5}{8}$" thick, for economy in chamfering and to reduce the width of the gap. Double V-butts may also be used for plates thinner than $\frac{5}{8}$" if it is not essential to grind the weld to a flush surface for bearing purposes. However, for plates from $\frac{3}{16}$" to $\frac{5}{8}$" thick, single V-butts are usually included, and are provided with a sealing run on the underside to ensure that the full sectional area of weld is obtained. The sealing run can, if desired, be dressed flush with the plate to give a level bearing surface. For general structural purposes, V-butts—either single or double—cover all normal requirements for butt welds.

For downhand welding, V-butts are the easiest of all welds to lay. They are so positioned that the molten metal assumes the desired shape by gravity. They are also comparatively cheap, as maximum welding speeds can be attained with this type of weld.

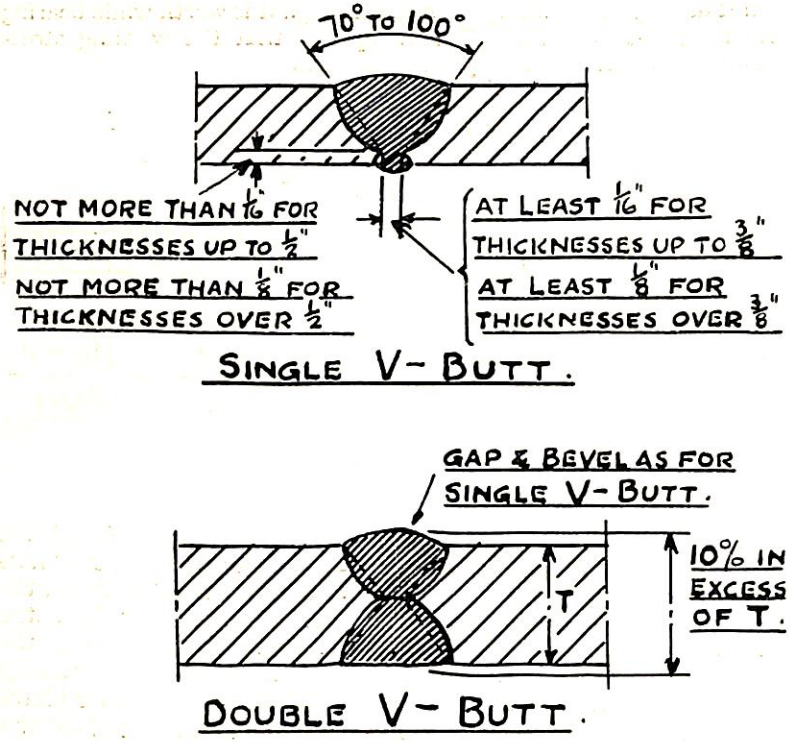
70° TO 100°
NOT MORE THAN 1/16 FOR THICKNESSES UP TO 1/2"
NOT MORE THAN 1/8" FOR THICKNESSES OVER 1/2"
AT LEAST 1/16" FOR THICKNESSES UP TO 3/8"
AT LEAST 1/8" FOR THICKNESSES OVER 3/8"
SINGLE V-BUTT.
GAP & BEVEL AS FOR SINGLE V-BUTT.
T
10% IN EXCESS OF T.
DOUBLE V-BUTT.

Fig. 7.

When for any reason it is not found possible to include a sealing run to a single V-butt, the working stress must be reduced by half, although this penalty is rather excessive.

U-Butt and J-Butt Welds. (Fig. 7A.)

These are very rarely used in structural practice unless special circumstances make it impossible to use a V-butt. The work of preparation is much greater than for a V-butt, with no corresponding advantage for normal work. U-butts or J-butts are necessary when it is not possible for the welder to see clearly the work in progress if a V-butt had been specified, or where a V-butt does not allow proper access for the electrode to the surfaces to be welded, but it is not often in normal structural practice that this is likely to occur. Rather than use these welds, it is much better to alter the detail to suit a V-butt.

Strength of Butt Welds.

L.C.C. Regulations allow a permissible stress in tension and compression of 8 tons per square inch. This means that a butt

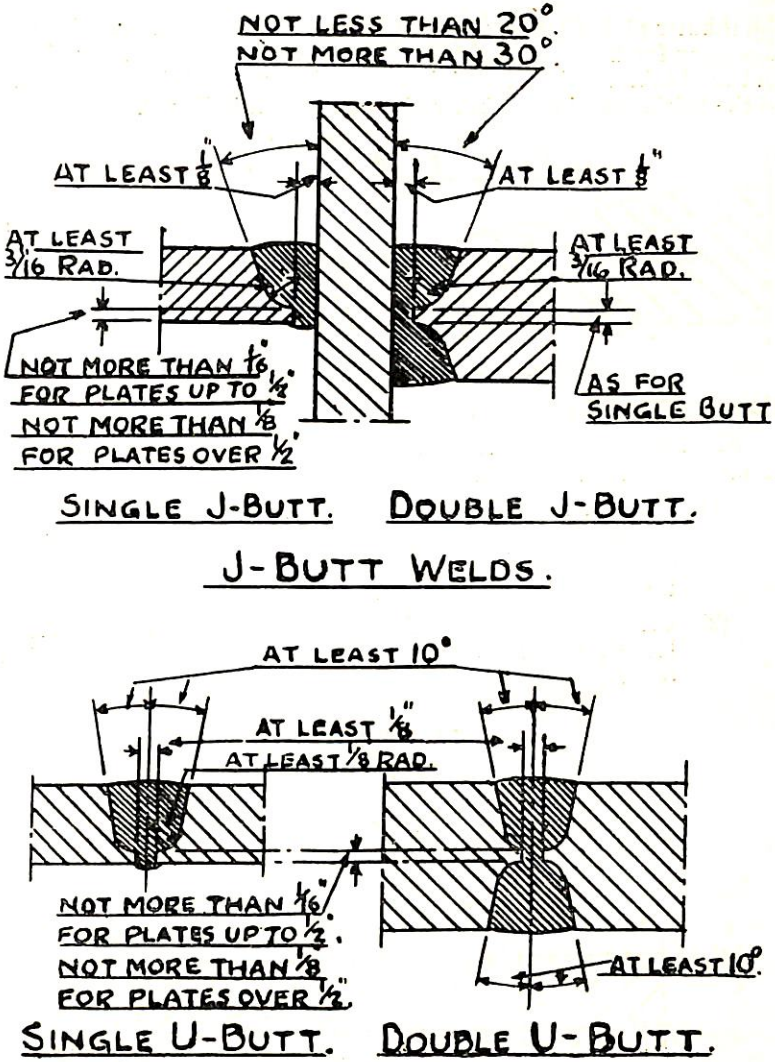

Fig. 7a.—J. & U. Butt Welds.

weld is assumed to have the same strength as the parent metal, and the throat thickness is taken as the thickness of the parent metal, or in the case of plates of different thickness, as the thickness of the thinner plate. No special calculations are, therefore, necessary for the strength of butt weld. Where two plates of unequal thicknesses have to be butt-welded, it is obvious that at the junction,

the thinner plate is strong enough to take the load, so that it is only necessary for the weld to develop the strength of the thinner plate, and to obtain a satisfactory joint, the thicker plate is bevelled down to the thickness of the thinner plate, as shown in Fig. 8.

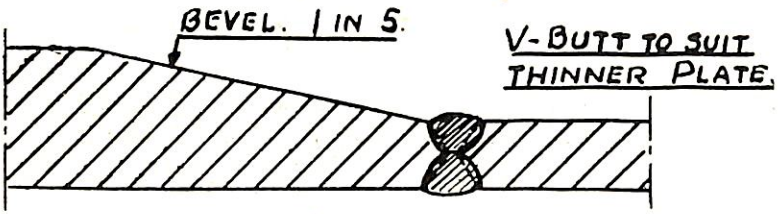

Fig. 8.—Plates of Unequal Thicknesses.

Horizontal and Vertical Fillets.

Fillet welds between horizontal and vertical plates are difficult to lay as downhand welds. The main difficulties are :—

(1) Maintenance of correct weld contour, due to the weld metal tending to flatten out on the horizontal plate, thus decreasing the throat thickness and therefore reducing the effective strength of the weld.

(2) Undercutting the vertical plate, thus creating a definite line of weakness at a point where stress concentration occurs. (See Fig. 9).

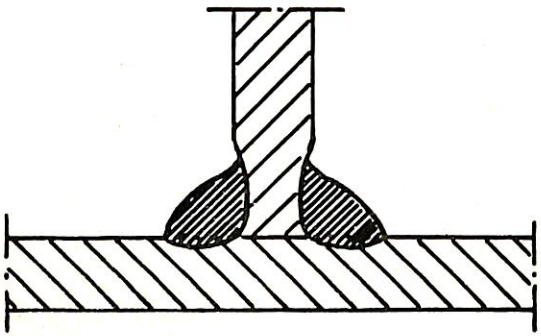

Fig. 9.—Undercutting of Vertical Plate.

In avoiding either of these errors it is quite possible that the welder may fall into the other one, and it is desirable when taking out electrode quantities to assume that to obtain a satisfactory job more weld metal will be deposited than is theoretically necessary. Where serious undercutting occurs, the reduction in area at that point must be made good by an additional run of weld metal.

Vertical Welding.

Vertical welds may be laid either downward or upward. The heat-absorbing property of the plates freezes the weld metal solid before it has had time to fall, providing the arc is not concentrated too long in any one locality. Surface tension also resists the tendency of the molten metal to fall. In downward welds only light runs should be attempted, or the molten metal will fall by gravity before it has had time to freeze, and any attempt to make a larger run only results in melting away metal already deposited. Downward welding is very suitable for sealing runs, and may also be used for seams up to $\frac{1}{4}$" or $\frac{5}{16}$".

Upward welding should be used for heavier seams, so that the deposited metal, when freezing, forms a ledge for succeeding runs.

For both upward and downward welding of vertical seams, the first run should be with an electrode of about 10 gauge to ensure good penetration.

Overhead Welding.

This is a most difficult operation, owing to the tendency of the molten metal to fall by gravity, and should only be attempted if no other procedure seems possible. The globules of metal passing across the arc must be frozen in the shortest possible space of time. Special "OVERHEAD" electrodes, possessing the necessary properties for quick freezing, are used for this work.

Overhead welding should always be in straight runs, and no weaving should be attempted.

Only welders of proved skill and efficiency should be employed in making these welds, and the welds should be laid in very short lengths, continually breaking the arc to enable the metal to freeze.

Inclined Welding.

The following procedure will usually be found to be satisfactory :

(1) For seams inclined to the horizontal at an angle of less than 45°, use downhand welds.
(2) For seams inclined to the horizontal at an angle greater than 45°, use upward vertical welds.
(3) For overhead seams inclined to the horizontal at an angle of less than 45°, use overhead welds.
(4) For overhead seams inclined to the horizontal at an angle greater than 45°, use upward vertical welds.

The above are only general directions, which are normally satisfactory, and each case must be judged on its merits.

Penetration and Fusion.

The weld profiles indicated in Figs. 1, 7 and 9 are not intended to show the actual amount of penetration into the parent metal,

or, of course, to infer that there is a line of demarcation between the deposited metal and the parent metal. The whole principle of welding is to produce a complete fusion of parent metal and weld metal, so that the pieces joined together become in effect one single piece, similar to a casting. For this purpose, a deep penetration is not necessary, and is in fact undesirable, as it is only necessary to melt the outside layers of crystals to obtain a proper fusion. Where there has been a deep penetration, it may mean that the welding current was unnecessarily high.

Distortion.

The distortion of component parts of a structure, produced chiefly by continuous welds, causes considerable difficulty. The distortion is due to the contraction of the molten metal on cooling, and unless adequate steps are taken to control this tendency, it is liable to pull the members out of shape to a prohibitive degree. In plates of large area, distortion is often caused by internal stresses set up during the process of manufacture, and obviously this is a point which it is difficult to detect. The methods generally adopted to control distortion may be summarised as follows :—

(1) If distortion seems likely to occur, the welding procedure must be designed to enable the pull of one weld to cancel out the pull of another weld.

(2) Individual runs should be made comparatively short, compared with the thickness of the metal, as distortion automatically increases as the weld lengthens.

(3) The parts should be rigidly tack welded together at intervals of 3-ft. to 4-ft. before any runs of weld are laid, in order to prevent displacement of individual pieces.

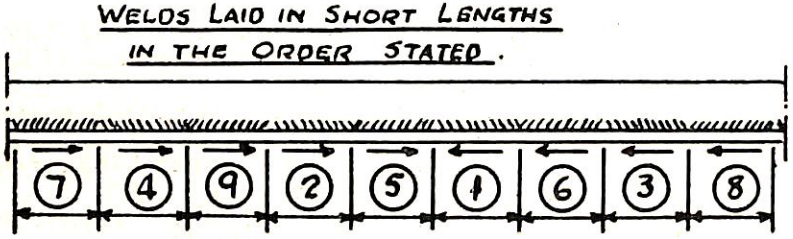

Fig. 10.—Typical Procedure to avoid Distortion.

(4) Welding may be in small sections, as in Fig. 10, so as to spread the heat fairly evenly about the work. Alternatively, the back-stepping method, shown in Fig. 11, may be used, or some variation of it. For intermittent welding, the procedure could be as shown in Fig. 12.

(5) The speed of travel during welding should be as rapid as possible, to prevent the heat moving ahead of the arc.

(6) The shape of the piece should be checked with a gauge from time to time during welding, and if distortion has commenced, the work must be stopped on the weld causing deformation, and an opposing weld laid to bring the piece back to correct profile.

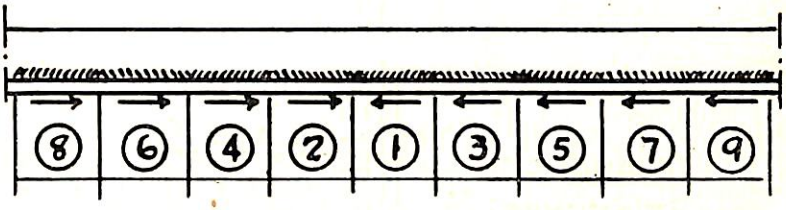

Fig. 11.—Alternative Arrangement of Welds (Back-Stepping) to avoid Distortion.

(7) Pre-heating of the plates is sometimes resorted to, so that the job expands and contracts as a whole, but this method is not sufficient in itself without additional safeguards.

(8) If it is expected that welding may produce distortion in any one direction, the plates may previously be given an initial distortion in the opposite direction, so that the welding will then pull them back to the correct shape. This is a somewhat rough-and-ready procedure, which may, however, be satisfactory where absolute accuracy of profile is not essential.

(9) When there is reason to believe that internal stresses have been set up in large plates during manufacture, distortion due to this cause may be prevented by re-rolling the plates before welding.

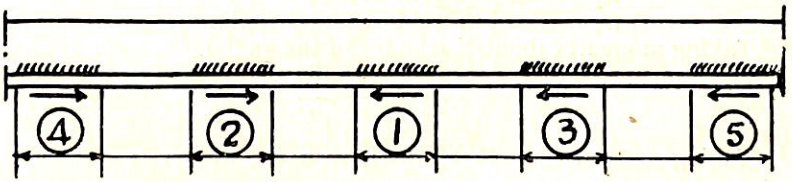

Fig. 12.—Arrangement of Intermittent Welds to avoid Distortion.

From the above remarks, it will be clear that distortion provides a major problem which is not easily solved. It will also be apparent that welded work requires a great deal more expert supervision than riveted work, and this point must always be taken into account in assessing relative costs.

Fig. 13 shows a method of holding the legs of angles in alignment during welding. When plate A is welded on, the angle may be distorted, and this can be prevented by first welding in the triangular stiffening plate B. Alternatively, if the angle is large, distortion may be obviated by using the back-stepping method of welding for plate A, but this procedure is not so satisfactory as the inclusion of the stiffening plate.

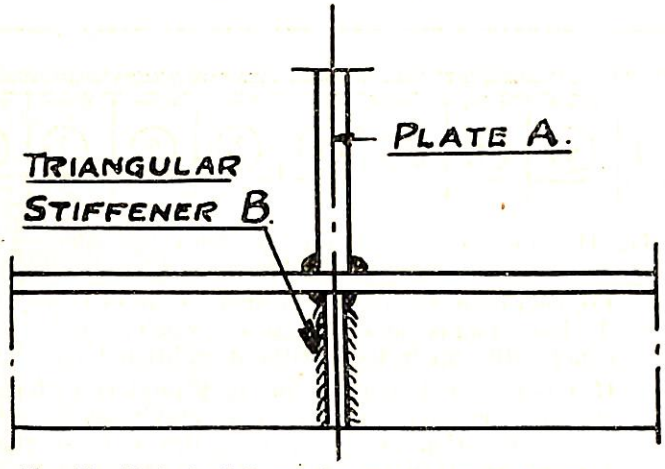

Fig. 13.—Method of Preventing Distortion in Angles.

Connection of Angles.

Case 1.—Full Area Effective.

Referring to Fig. 14,

Let A = area of section of angle.
f = permissible stress in angle.
P = load carried by angle = Af
S_1 = load in weld at heel of angle.
S_2 = load in weld at toe of angle.

Taking moments about X (the toe of the angle),

$$S_1 = \frac{P\,b}{a+b}$$

Taking moments about Y (the heel of the angle),

$$S_2 = \frac{P\,a}{a+b}$$

Using fillets of equal size, of strength f_1 tons per lineal inch.
Length of weld,

$$S_1 = L_1 = \frac{S_1}{f_1} = \frac{P\,b}{f_1\,(a+b)} = \frac{P}{f_1}\left(\frac{b}{a+b}\right)$$

Length of weld,

$$S_2 = L_2 = \frac{S_2}{f_1} = \frac{P\,a}{f_1\,(a+b)} = \frac{P}{f_1}\left(\frac{a}{a+b}\right)$$

If for constructional reasons it is desirable to have equal lengths of weld at heel and toe, then,

Strength per lineal inch required at heel

$$= \frac{S_1}{L} = \frac{P\,b}{L\,(a+b)} = \frac{P}{L}\left(\frac{b}{a+b}\right)$$

Strength per lineal inch required at toe

$$= \frac{S_2}{L} = \frac{P\,a}{L\,(a+b)} = \frac{P}{L}\left(\frac{a}{a+b}\right)$$

From the above calculation it will be seen that whether it is desired to find the lengths of equal welds, or the size of weld for equal lengths, in each case the required values are in the same ratio to each other as dimension b is to dimension a.

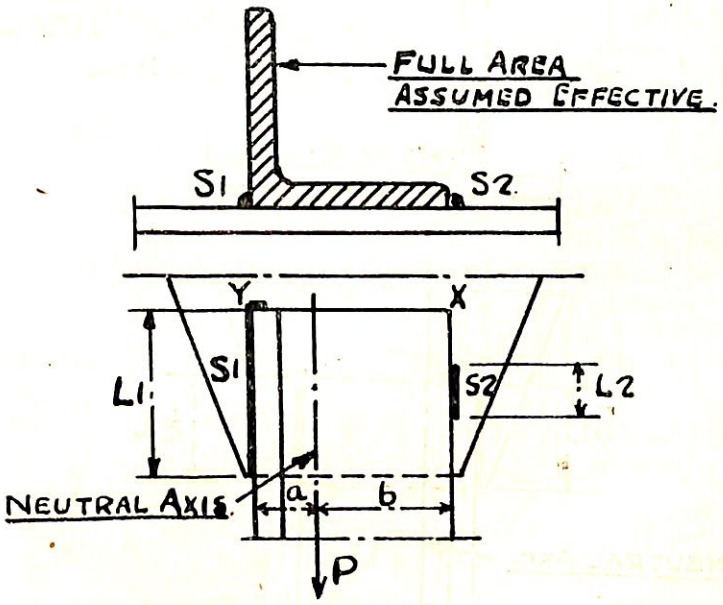

Fig. 14.—Connection of Angles.

Connection of Angles.

Case 2.—Assuming only half of the area of the Outstanding Leg is effective.

Referring to Fig. 15.

Let A = assumed area of angle.

= area of horizontal leg + half area of vertical leg.
f = permissible stress in angle.
P = load carried by angle = Af.

Using fillets of equal size, of strength f_1 tons per lineal inch.

Nett length of weld on each side of angle = $P/2f_1$

This procedure is to some extent approximate, as it assumes that the effective area of the angle is divided into equal parts by the neutral axis. The degree of error is very small, especially on small angles, but if it is desired to be exact, the neutral axis of the effective area must be found, and the procedure outlined in Case 1 followed.

If it is not possible to obtain a sufficient length of side fillet, an end fillet must also be introduced. Fig. 38, showing details of a welded roof truss, is a typical example of the type of connections usually employed in frames built up of small sections.

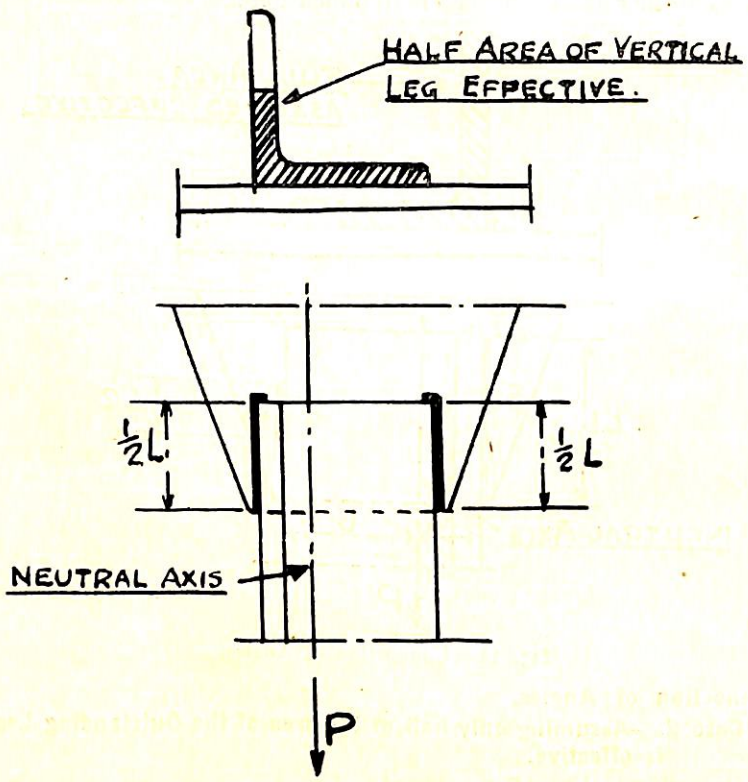

Fig. 15.—Connection of Angles.

Beam Seatings on Stanchions.

For riveted construction, a seating consists of either a cleat or a built-up bracket, according to the load on the seating. If the stanchion were a casting, the seating would be a lug cast on, with or without a stiffener to suit the load. Maintaining the principle that welded work is analogous to a casting, rather than to riveted work, a lug consisting of a flat about $\frac{1}{2}''$ thick, as shown in Fig. 16 (a), can be welded on if the seating is purely a rest taking no load. If the seating transmits load, then a tee or a joist cutting

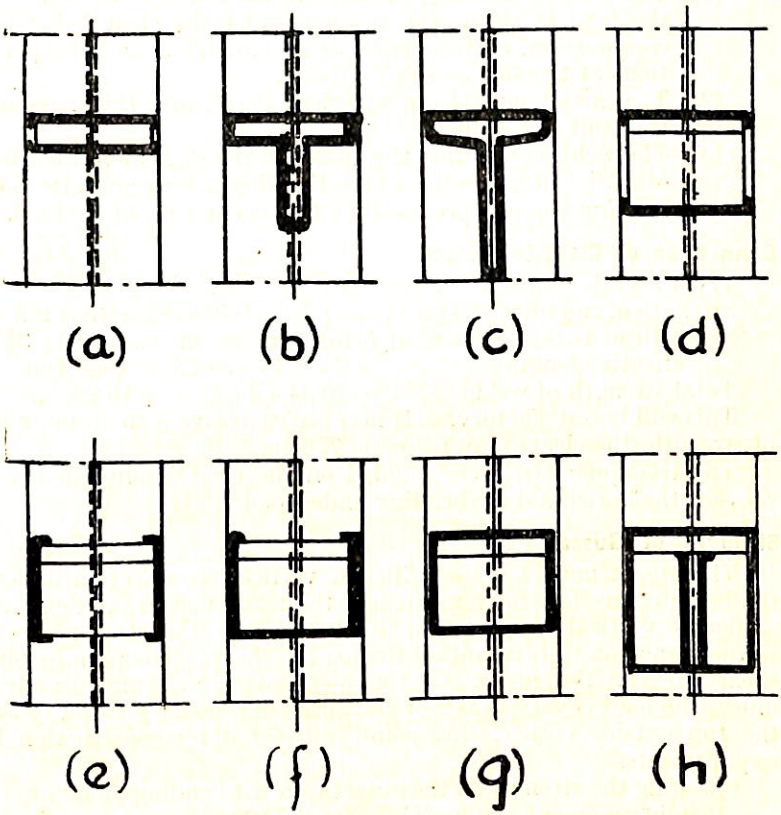

Fig. 16.—Beam Seatings.

may be used, as in Figs. 16 (b) and (c). Figs. 16 (d), (e), (f) and (g) show cleats with either side or end fillets, or with both, according to load carried, and Fig. 16 (h) shows a cleat with a stiffener. In the case of the horizontal fillets on all these connections, where the fillets are in compression (on the underside of the cleat or lug) they may be designed as end fillets at 6 tons per square inch, but where

they are in tension (on the top of the cleat or lug) they should be designed as side fillets at 5 tons per square inch. The more usual procedure, however, is to take the total effective length at 5 tons per square inch.

Eccentric Beam Seatings.

Referring to Fig. 17, the load must be transferred through the cleat into the gusset, and thence to the stanchion flange. There are thus three stages in design :—

(1) The welds connecting the cleat to the gusset must transfer the load in shear, the portion under the cleat being in compression, and designed as an end fillet, and the portions at the side as side fillets.

(2) The gusset must have sufficient depth and thickness to transmit the load.

(3) The welds connecting the gusset to the stanchion must be of sufficient strength to take the direct shear and also the bending moment produced by the eccentricity of the load.

Connection of Cleat to Gusset.

Try $4 \times 4 \times \frac{1}{2}$" cleats, $6\frac{1}{2}$" long, with $\frac{3}{8}$" fillets.

Strength of end fillet at 6 tons per sq. in. $= 1{\cdot}59 \times 6{\cdot}5 = 10{\cdot}34$ tons.

Strength of two-side fillets at 5 tons per sq. in. (assuming $3\frac{5}{8}$" effective length) $= 2 \times 1{\cdot}33 \times 3{\cdot}625 = 9{\cdot}64$ tons.

Total strength of welds $= 10{\cdot}34 + 9{\cdot}64 = 19{\cdot}98$,,

This will be satisfactory. If necessary, to give a small margin of strength, the cleats may be made 7" long.

The triangular stiffener included on the cleat is intended to prevent the horizontal leg bending under load.

Strength of Gusset.

The gusset must have a sufficient vertical cross-section under the beam centre line to transmit half the load in shear, and also a sufficient vertical cross-section at the edge of the cleat nearest to the stanchion to transmit all the load in shear. The permissible shear stress in the gusset is 5 tons per square inch, and usually, unless the load is very heavy or the gusset has a sharp cutaway at the top outside corner, this point is covered by constructional requirements.

Checking the strength of the gusset to resist bending moment,

Bending moment at line YY $= 20 \times 6 = 120$ tons-ins.

Permissible bending stress in gusset $= 8$ tons per sq. in.

Z required in gusset at YY $= 120/8 = 15$.

$$Z \text{ given} = \frac{BD^2}{6} = \frac{B \times 12 \times 12}{6} = 24\,B$$

$\therefore$ B (thickness) $= 15/24 = 0{\cdot}625$.

Use $\frac{5}{8}$" thick gusset.

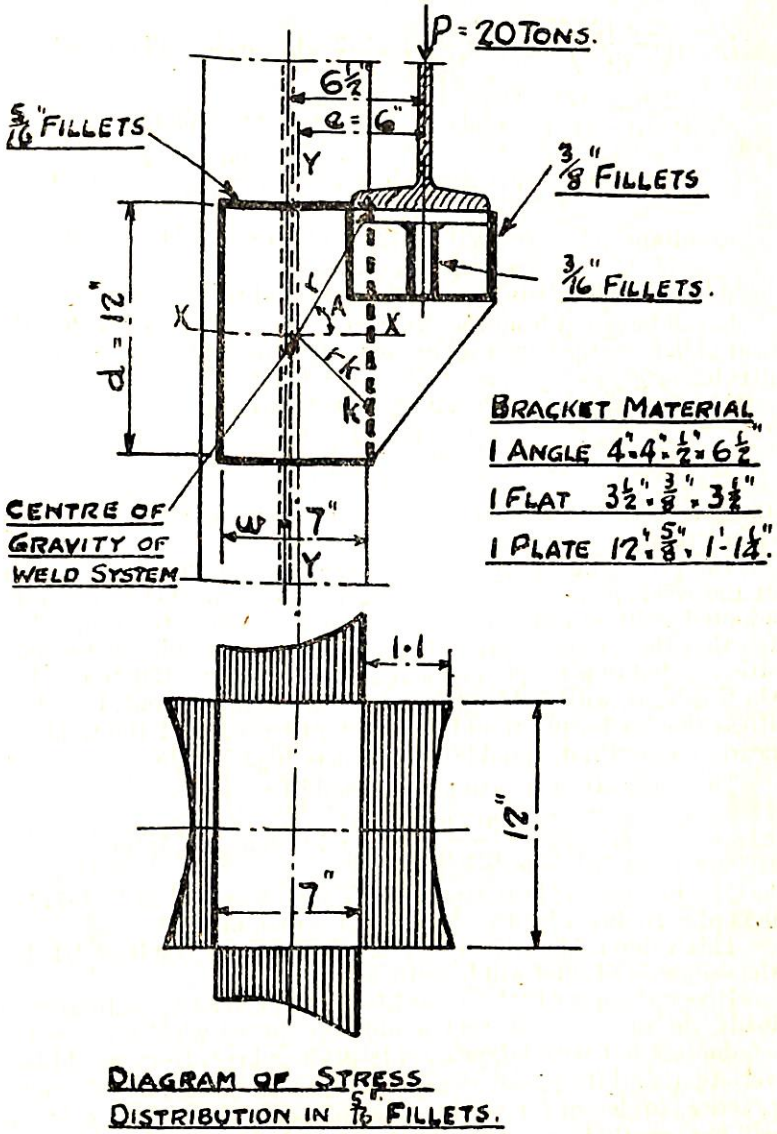

Fig. 17.—Eccentric Beam Seating.

Welds Connecting Gusset to Stanchion.

$P = 20$ tons, $e = 6''$, $w = 7''$, $d = 12''$, $r = \sqrt{3\frac{1}{2}^2 + 6^2} = 7''$.

Total length of fillet weld $= 2\,(7 + 12) = 38''$.

I_p = polar moment of inertia $= I_{xx} + I_{yy}$. For 1" throat thickness of weld

$$Ip = \left(\frac{2 \times 12^3}{12}\right) + \left(\frac{2 \times 7^3}{12}\right) + (2 \times 12 \times 3\tfrac{1}{2}^2) + (2 \times 7 \times 6^2)$$

$$= 288 + 57 + 294 + 504 = 1143.$$

Shear stress (F*s*) per inch of weld = 20/38 = 0·53 ton.

$$\text{Bending stress (F}b\text{) per inch of weld} = \frac{20 \times 6 \times 7}{1143} = 0{\cdot}74 \text{ ton.}$$

$$\text{Resultant } F_R = \sqrt{0{\cdot}53^2 + 0{\cdot}74^2 + (2 \times 0{\cdot}53 \times 0{\cdot}74 \times \cos A)}.$$

$$= 1{\cdot}1 \text{ tons per inch.}$$

$\frac{5}{16}$" fillet will satisfy this requirement and will be used.

It will be noted from the diagram of stress distribution that the load of 1·1 tons per inch occurs only at two points, and elsewhere it is less, *e.g.* :—

At any point K, the bending stress F*b* will be

$$\left(\frac{20 \times 6 \times rk}{1143}\right)$$

and this value is less than the maximum bending stress previously calculated.

The procedure given above for the calculation of the stresses in the welds connecting the gusset to the stanchion is generally adopted in structural practice, but is open to some criticism. It is felt that these stresses are not bending stresses at all, but torsional stresses, due to a torque moment equal to $20 \times 6 = 120$ ton-inches. On this basis, with welds of equal thickness throughout, the shear stress due to torque would be constant throughout the welds, or nearly so, and not variable as shown in Fig. 17.

The shear stress due to torque would be

$$fs = \frac{20 \times 6}{2 \times 12 \times 7 \times 1} = 0{\cdot}714 \text{ tons per sq. in.}$$

The maximum shear stress in the weld would then be 0·714 + 0·53 (due to direct load) = 1·244 tons per sq. in.

This value is slightly more than the previous result of 1·1, but the same size of fillet will be sufficient.

It would appear that for most brackets of this type with average loads, the same size of weld would be required whichever method is adopted, but with larger eccentricities or loads there would be a variation, and it appears desirable, in the present stage of welding practice, to design for safety, as an increase of $\frac{1}{16}$" in a weld size will not greatly increase the cost.

Eccentric Beam Seating (Special Case).

Fig. 18 is generally similar in construction to Fig. 17, the only difference being that the gusset is welded to the stanchion flange on three sides only. The centre of gravity of the weld system is

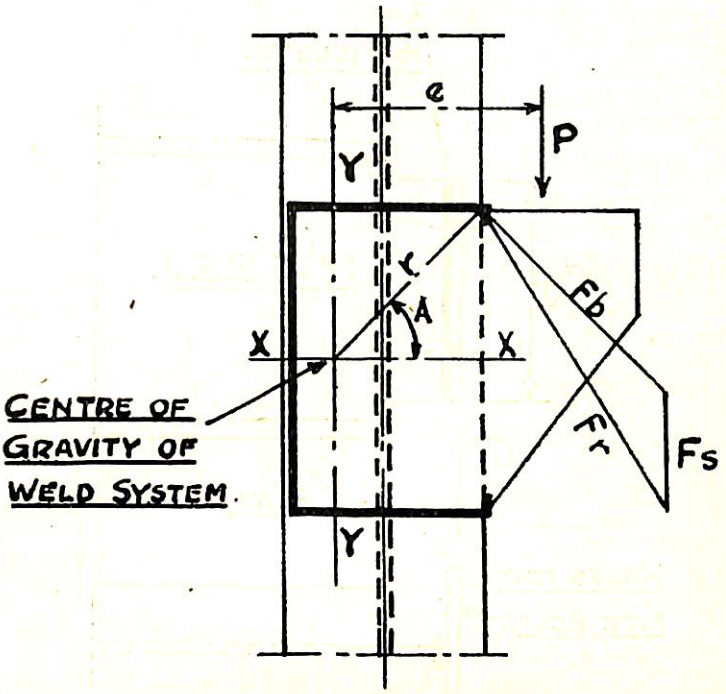

Fig. 18.—Eccentric Beam Seating.
Special Case with Welds on 3 Sides only.

first found, and the procedure is then exactly as for Fig. 17. It may be noted that the eccentricity for this type of connection is greater than that shown in Fig. 17, so that the bending moment will also be greater, and a deeper bracket will be required for the same load.

Beam Connections (Non-Rigid).

Fig. 19 illustrates a typical beam end detail, which is suitable for practically any beam section. This detail compares very favourably with a riveted connection. There is very little eccentricity of load on the welds, which may be designed as side fillets for ordinary shear conditions.

Depth of web at end = 9″
Reaction = 6 tons.
Assume effective depth of weld = $8\frac{1}{2}$″

Load per inch of weld $= \frac{6}{8\frac{1}{2} \times 2} = 0{\cdot}352$ ton.

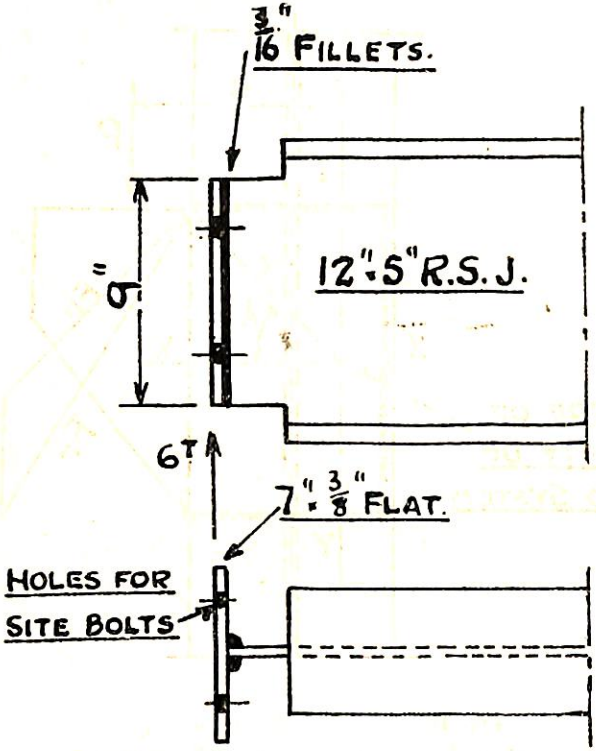

Fig. 19.—Beam Connections (Non-Rigid) Shop Welded.

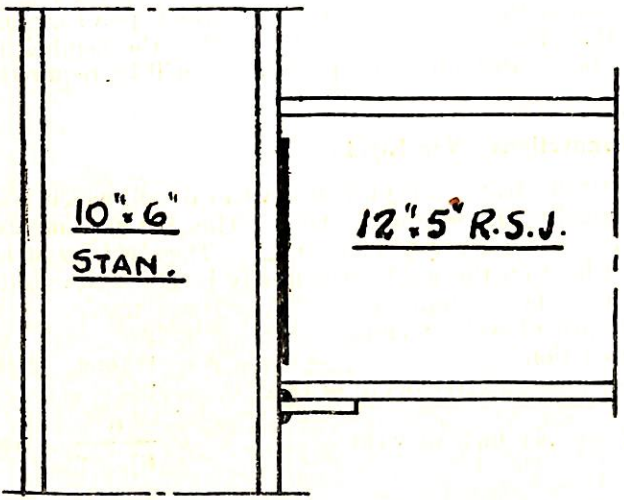

Fig. 20.—Beam Connections (Non-Rigid) Site Welded.

$\frac{3}{16}$" fillets are ample and will cover for any slight effect of eccentricity. The end plate will be made the same thickness as the web of the beam, say $\frac{3}{8}$", and of a sufficient width to get in the site bolts.

For a deep beam with a small reaction, it is not essential to make the welds continuous. These may be intermittent, with a sufficient total effective length to take the reaction.

Fig. 20 shows an alternative type of beam connection, involving site-welding. Site-welding is nearly always a hazardous and expensive operation, particularly when carried out in the air, as in this case, and there appears to be no adequate reason for preferring this type of connection to that shown in Fig. 19. The lug in the stanchion serves purely as a rest, and the fillets to the beam web are designed to take the reaction as a simple shear connection, as in the case of Fig. 19.

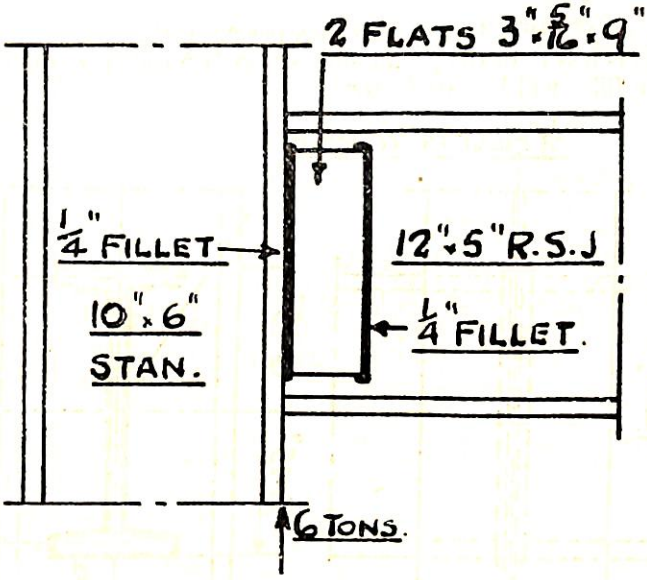

Fig. 21.—Beam Connections (Non-Rigid) Site Welded.

A further alternative end detail is shown in Fig. 21, which also incorporates site welding. The flats in the web of the beam are shop-welded to the beam, and site-welded to the stanchion. It is assumed that the welds share the bending moment produced by the eccentricity of the connection.

Let t = throat thickness of weld.
Z = section modulus of weld.
F_b = stress in weld due to B.M.
F_s = stress in weld due to shear.
F_r = resultant stress in weld.

Bending moment = $6 \times 3 = 18$ tons inches.
Bending moment taken by each pair of welds = $18/2 = 9$ tons inches.
Direct shear = 6 tons.
Shear per inch = $6/9 = 0{\cdot}67$ ton.

$$Z = \frac{2 \times t \times 9 \times 9}{6} = 27\,t$$

It is considered undesirable to use a fillet of less than $\frac{1}{4}$", and this size will be tried.

Z of $\frac{1}{4}$" fillet = $27 \times {\cdot}25 \times {\cdot}707 = 4{\cdot}73$
$F_h = 9/4{\cdot}73 = 1{\cdot}9$ tons per sq. inch.

$$F_s = \frac{0{\cdot}67}{0{\cdot}25 \times 0{\cdot}707} = 3{\cdot}8 \text{ tons per sq. inch.}$$

$F_r = \sqrt{1{\cdot}9^2 + 3{\cdot}8^2} = 4{\cdot}25$ tons per sq. inch.

This is less than the permissible stress of 5 tons per square inch, and the fillet will be satisfactory.

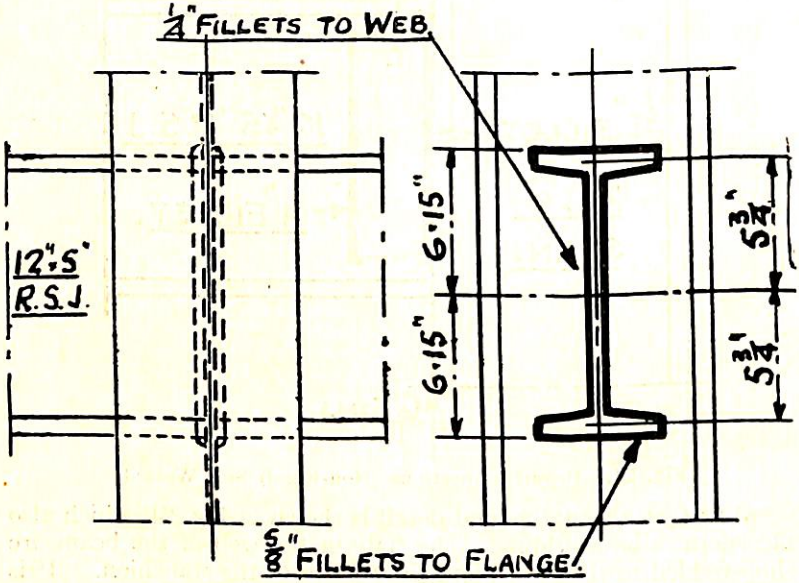

Fig. 22.—Beam Connection to Stanchion Web. (Rigid Type) Site Welded.

Rigid Beam Connections.

Type 1.—Beams to Stanchion Webs (Fig. 22).

The general principle of design is to make the welds as strong as the beam.

I of beam = 221.

Assume $\frac{5}{8}$" fillets to flange with throat thickness of 0·44", and $\frac{1}{4}$" fillets to web with throat thickness of 0·175."

I of flange fillets = $4 \times 0{\cdot}44 \times 5 \times 5\frac{3}{4} \times 5\frac{3}{4}$ = 290·1

I of web fillets = $\dfrac{2 \times 0{\cdot}175 \times 10 \times 10 \times 10}{12}$ = 29·2

I of welds = 290·1 + 29·2 = 319·3

Moment of resistance of beam = $\dfrac{f\,I}{y} = \dfrac{8 \times 221}{6}$ = 294·7

Moment of resistance of welds = $\dfrac{f\,I}{y} = \dfrac{6 \times 319{\cdot}3}{6{\cdot}15}$ = 311·5

The welds are, therefore, slightly stronger than the beam, and the connection is satisfactory.

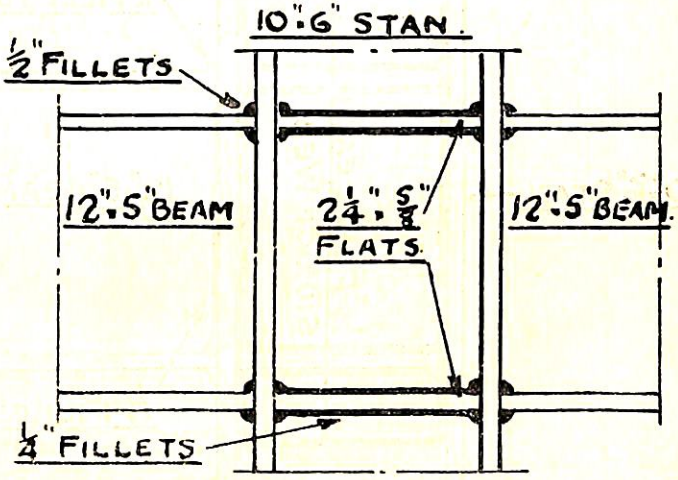

Fig. 23.—Beam Connection to Stanchion Flanges. (Rigid Type) Site Welded.

Type 2.—Beams to Stanchion Flanges (Connections on both flanges). Fig. 23.

The procedure is to assume that the maximum permissible bending moment is developed at the connection, with a maximum working stress of 8 tons per square inch. The connection is then as strong as the beam.

Thickness of flange = 0·55"
Tension in top flange = $5 \times 0{\cdot}55 \times 8$ = 22 tons.
Compression in bottom flange = 22 tons.

These forces must be carried through the stanchion. Load per lineal inch of flange weld = $22/(5 \times 2)$ = 2·2 tons.

$\frac{1}{2}$" end fillets at 6 tons per square inch will take 2·12 tons per lineal inch, and may be assumed to be sufficient. The flanges of the stanchion require stiffening to prevent deformation, and to assure uniformity of weld stress.

Load in stiffening flats = 22/2 = 11 tons.

(Assuming all load is transmitted through flange to flat).

Area required in flat = 11/8 = 1·38 sq. inches.

Use $2\frac{1}{4}" \times \frac{5}{8}"$ flats.

The loads in the welds at each end of each stiffening flat will be 11 tons, and $\frac{1}{2}$" end fillets will again be used. The stiffening flats require tack welding to the stanchion web, and $\frac{1}{4}$" fillets will be suitable for this purpose.

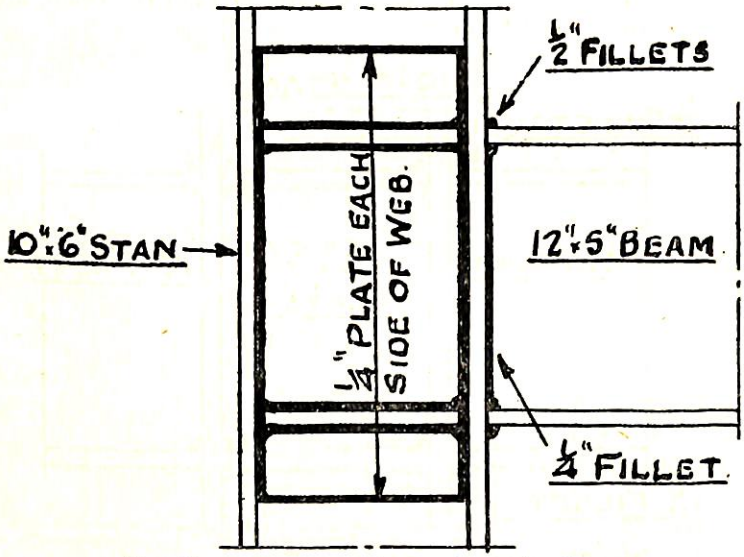

Fig. 24.—Beam Connection to Stanchion Flange. (Rigid Type) Site Welded.

Type 3.—Beam to Stanchion Flange. (One side only).

This case, which is shown in Fig. 24, is generally similar to Type 2 and requires similar treatment, except that the stanchion now requires stiffening to resist buckling.

Load from 12 × 5 joist flange (as before) = 22 tons.

$$\text{Shear in web of stanchion} = \frac{22}{8 \times 0{\cdot}4} = 6{\cdot}9 \text{ tons per sq. inch.}$$

(Assuming all load is transmitted to web).

Allowable buckling stress = $(5{\cdot}5 - 0{\cdot}04\ d/t) = 4{\cdot}5$ tons (where d = nett depth and t = thickness of stanchion web).

The web is therefore too weak and must be stiffened. $\frac{1}{4}$" flats will be sufficient for this purpose, and these must be securely welded to the profile of the stanchion flange, and also to the web.

Shear in web now equals $\frac{22}{8 \times 0{\cdot}9} = 3$ tons per sq. inch.

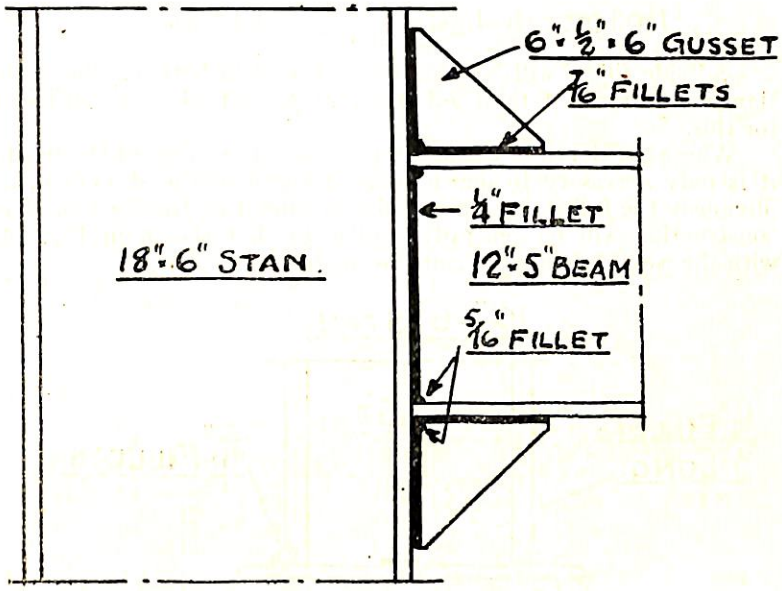

Fig. 25.—Beam Connection to Stanchion Flange. (Rigid Type) Site Welded.

Where the depth of the stanchion is considerably more than that of the beam, as in Fig. 25, a slightly different treatment is called for. In this case the stanchion web has ample area to resist buckling and does not need reinforcement. The stiffeners to the stanchion flange may also be superseded by the gusset construction shown. The beam, together with the gussets, may now be regarded as a steel casting, and loaded somewhat similarly to a column subject to bending moment. It is clear that part of the load from the flanges will be transferred to the 18×6 joist through the web of the 12×5, and it would not be unreasonable to assume that the web accounts for half of the load. Assuming for safety that the web takes only one-third, and the gussets take two-thirds of the flange load, then

Load transferred through gusset $= \frac{2}{3} \times 22 = 14{\cdot}7$ tons.
" " " web $= \frac{1}{3} \times 22 = 7{\cdot}3$ tons.
Gussets are made $6'' \times \frac{1}{2}'' \times 6''$ and sectional area $= 6 \times \frac{1}{2} \times 0{\cdot}7 = 2{\cdot}1$ sq. inches.
Stress in gusset $= 14{\cdot}7/2{\cdot}1 = 7$ tons per sq. inch.
If the full strength of the gusset is to be developed in the weld, then

$$\text{Load on weld} = 2{\cdot}1 \times 8 = 16{\cdot}8 \text{ tons.}$$

$$\text{Load per inch of weld} = \frac{16{\cdot}8}{6 \times 2} = 1{\cdot}4 \text{ tons.}$$

$\frac{7}{16}''$ side fillets will be suitable. The weld between the beam flange and the 18×6 takes 7·3 tons, and $\frac{5}{16}''$ end fillets are sufficient for this.

Where the depth of the stanchion is less than that of the beam, it is only necessary to develop the strength of the stanchion, as obviously the full strength of the beam cannot be transferred. The construction will be generally similar to that shown in Fig. 24, with the welds modified to suit the conditions.

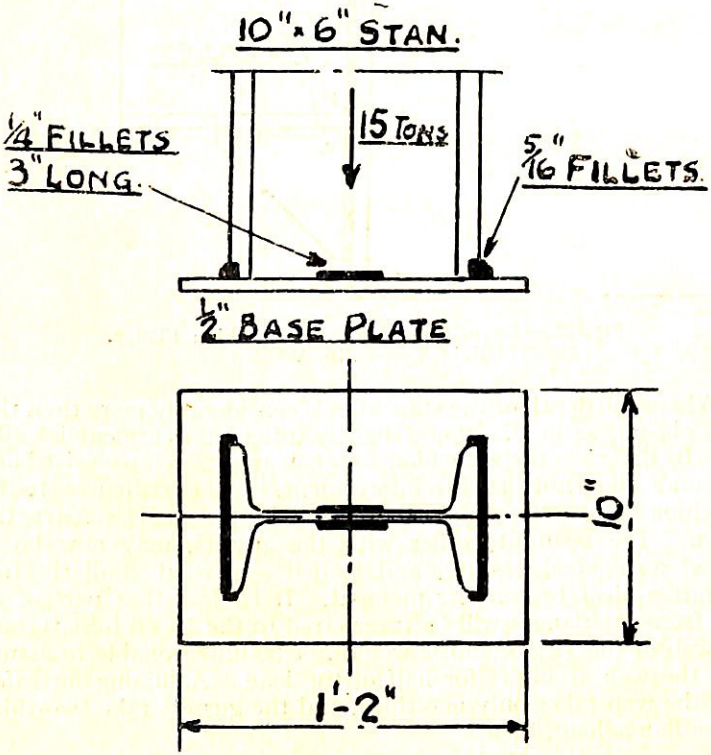

Fig. 26.—Stanchion Base for Small Vertical Load.

Assume the stanchion is of 8×5 joist section.

$$Z \text{ of } 8 \times 5 = 22{\cdot}42$$

Moment of resistance at 8 tons per square inch

$$= 22{\cdot}42 \times 8 = 179{\cdot}36 \text{ tons inches.}$$

The maximum bending moment which can be introduced into the stanchion will be 179·36 tons inches.

Load carried by flange welds = 179·36/11·5 = 15·6 tons.

Load per inch of weld = 15·6/10 = 1·56 tons.

$\frac{3}{8}$" end fillets will be sufficient.

As in the previous case, the stanchion web will require reinforcement to resist buckling and stiffeners will also have to be introduced between the stanchion flanges, capable of taking the

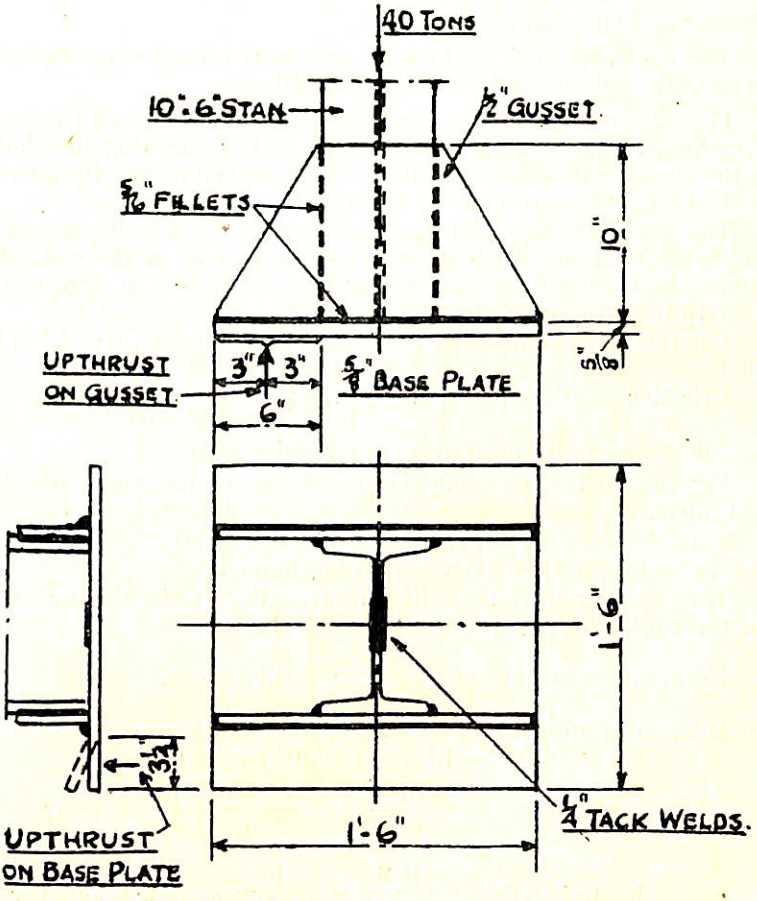

Fig. 27.—Stanchion Base for Larger Vertical Load.

permissible load of 15·6 tons, the procedure being as previously outlined.

Stanchion Bases. (Subject to vertical load only).

The procedure is generally similar to that for beam connections of the rigid type. Fig. 26 shows a typical base for a stanchion carrying a small load and with all the load assumed to be transferred through the flange welds. The welds are returned round the profile of the flange, giving an effective length of weld of 6″ per flange.

$$\text{Load per inch of weld} = \frac{15}{6 \times 2} = 1{\cdot}25 \text{ tons.}$$

$\frac{5}{16}$″ end fillets will be sufficient.

The short tack welds to the web are assumed to act as positioning welds only, and are not included in calculations.

Fig. 27 shows a base for a heavier load. Assuming all the load is transferred through the gussets, the welds connecting the shaft to the gussets will act as side fillets, and those connecting the gusset to the base plate as end fillets, as they are in compression.

The size of the base plate is fixed to suit the allowable pressure on the foundation, which in turn will depend upon the concrete mix. The base plate shown will suit an allowable pressure of at least 18 tons per square foot.

For the fillets connecting the shaft to the gussets, try $\frac{5}{16}$″ side fillets—

$$\text{Effective length required in each fillet} = \frac{40}{4 \times 1{\cdot}1} = 9{\cdot}1''$$

Allow gussets 10″ deep to cover for end craters.

For the fillets connecting the gussets to the baseplate, try $\frac{5}{16}$″ end fillets.

Total effective length required = 40/1·31 = $30\frac{1}{2}$″.

36″ total length is given, and will be ample.

The gussets must have sufficient strength to resist the upthrust on the cantilevered portion outside the shaft.

$$\text{Upthrust on two gussets} = \frac{40 \times 6}{18} = 13{\cdot}33 \text{ tons.}$$

Moment of upthrust about edge of joist flange

$$= 13{\cdot}33 \times 3 = 40 \text{ tons inches.}$$

$$Z \text{ of two gussets} = \frac{2\,BD^2}{6} = \frac{2 \times \frac{1}{2} \times 10 \times 10}{6} = 16{\cdot}67$$

$$\text{Stress due to bending} = \frac{\text{B.M.}}{Z} = \frac{40}{16{\cdot}67} = \pm 2{\cdot}4 \text{ tons per sq. inch.}$$

This stress is purposely kept low, as the sloping edge of the gusset is in compression.

The base plate thickness must also be checked to resist upthrust on the projecting portion. The projection outside the weld is taken at $3\frac{1}{4}$", and the load on this portion is

$$\frac{40 \times 3\frac{1}{4}}{18} = 7{\cdot}2 \text{ tons.}$$

$$\text{Bending moment of upthrust} = \frac{7{\cdot}2 \times 3\frac{1}{4}}{2} = 11{\cdot}7 \text{ tons inches.}$$

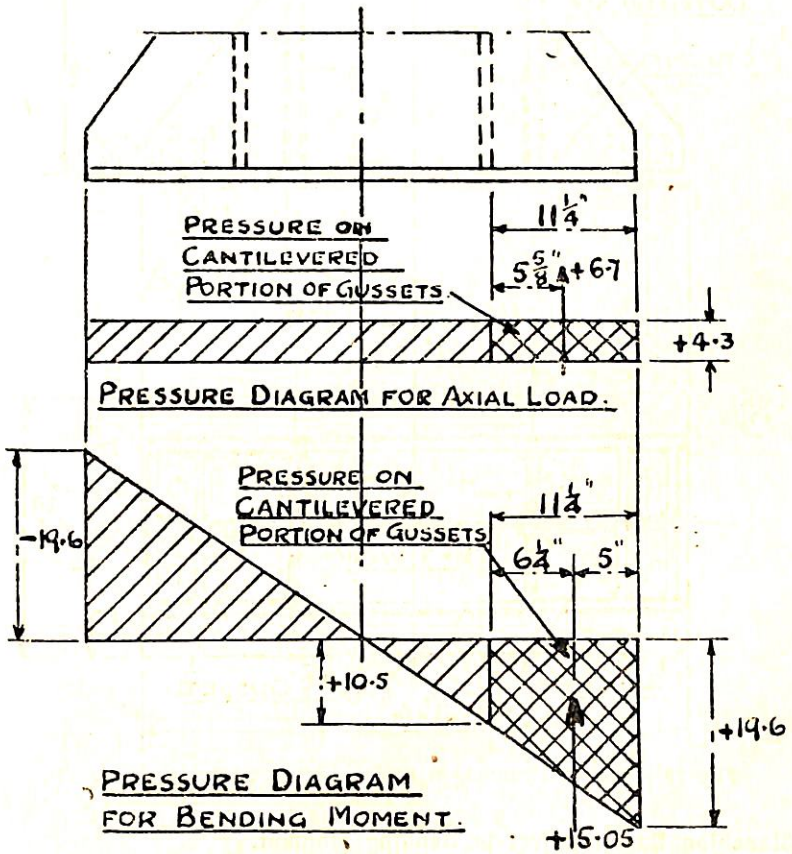

Fig. 28.—Pressure Diagram for Base Plate, shewing Upthrust on Gussets.

$$Z \text{ of base plate} = \frac{BD^2}{6} = \frac{18 \times \frac{5}{8} \times \frac{5}{8}}{6} = 1{\cdot}17$$

$$f = \frac{\text{B.M.}}{Z} = \frac{11{\cdot}7}{1{\cdot}17} = 10 \text{ tons per square inch.}$$

A stress of 10 tons in base plates is not considered excessive, and the thickness of $\frac{5}{8}$" will be suitable.

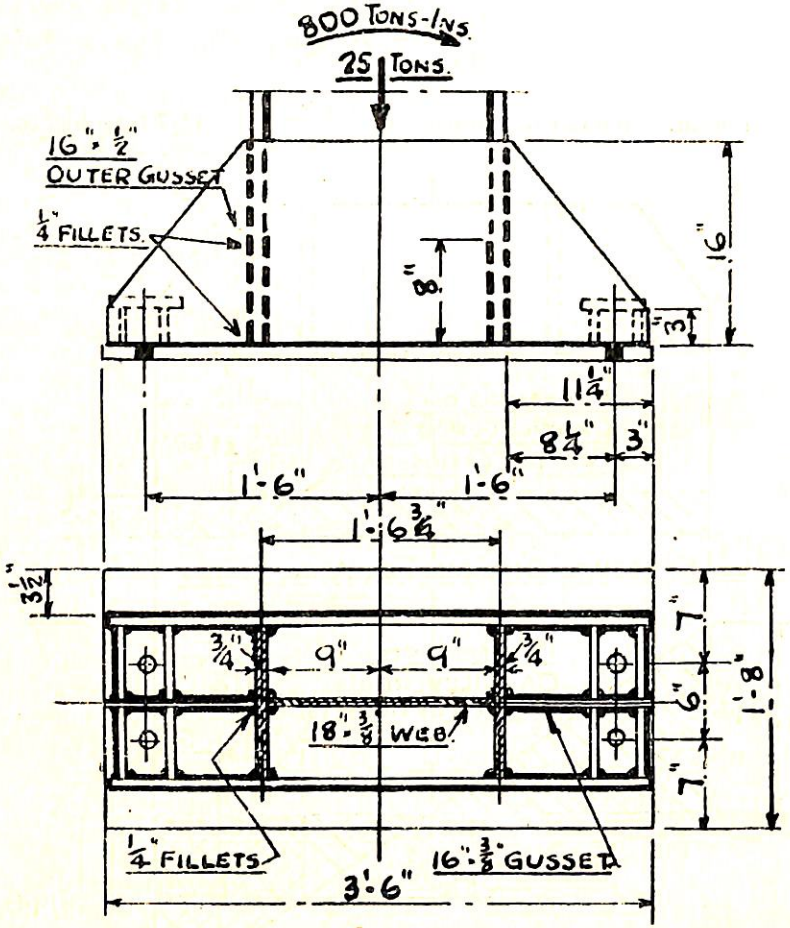

Fig. 29.—Detail of Stanchion Base taking Bending Moment and Axial Load.

Stanchion Bases subject to Bending Moment.

Fig. 29 shows a typical welded steel base. The stanchion shaft is composed of two 12" × ¾" flange plates and an 18" × ⅜" web plate. A section of this kind is equivalent to a broad flange beam, and is very economical for stanchions subject to bending moment, as it

throws the bulk of the metal into the flanges, where it is most needed. A riveted section of equal strength would be built up with a joist and flange plates, so that the welded construction eliminates all workmanship in drilling and riveting the flanges. It is not necessary to continue the $12 \times \frac{3}{4}$ flanges to the full height of the stanchion. The flange section can be reduced at some convenient point to suit the reduced bending moment, the joint being made by means of a butt weld.

The axial load on the stanchion base is 25 tons, and the bending moment at the base is 800 tons inches. The allowable pressure on the concrete foundation is 25 tons per square foot.

Area of base plate $= 3{\cdot}5 \times 1{\cdot}67 = 5{\cdot}83$ sq. feet.

Pressure on foundation due to axial load $= 25/5{\cdot}83$
$= 4{\cdot}3$ tons per sq. foot.

Allowable pressure remaining to take bending moment
$= 25 - 4{\cdot}3 = 20{\cdot}7$ tons per sq. foot.

$$Z \text{ required in base plate (in feet units)} = \text{B.M.}/f = \frac{800}{12 \times 20{\cdot}7} = 3{\cdot}22$$

$$Z \text{ given (in feet units)} = BD^2/6 = \frac{5 \times 7 \times 7}{3 \times 2 \times 2 \times 6} = 3{\cdot}4$$

The base plate size is therefore satisfactory. Actual maximum stress in foundation due to B.M.

$$= \text{B.M.}/Z = \frac{800}{12 \times 3{\cdot}4} = 19{\cdot}6 \text{ tons per sq. foot.}$$

The holding-down bolts must be designed on the assumption that rotation takes place about the leeward bolts.

Let T = tension in one bolt.
W = axial load.
D = bolt centres parallel to direction of wind.

Then $(2\,T \times D) + (W \times \frac{1}{2}\,D) = \text{B.M.}$
$(2\,T \times 36) + (25 \times 18) = 800$
$T = 4{\cdot}86$ tons.

The side thrust on the stanchion due to wind load was 5 tons, so that—

Shear per bolt due to side thrust $= 5/4 = 1{\cdot}25$ tons.
Try $1\frac{1}{8}''$ diameter bolts.
Area at bottom of thread $= 0{\cdot}7$ sq. ins.
Area of shank $= 1{\cdot}0$ sq. in.
Tensile stress (f_t) per square inch $= 4{\cdot}86/0{\cdot}7 = 6{\cdot}94$ tons.
Shear stress (f_s) per square inch $= 1{\cdot}25/1{\cdot}0 = 1{\cdot}25$ tons.

From the Theory of Principle Stresses—

$$\text{Maximum shear stress } V = \sqrt{\left(\frac{f_t}{2}\right)^2 + f_s^2}$$

$$= \sqrt{3{\cdot}47^2 + 1{\cdot}25^2} = 3{\cdot}7 \text{ tons per sq. in.}$$

Maximum tensile stress $F = \frac{f_t}{2} + V = 3{\cdot}47 + 3{\cdot}7$
$= 7{\cdot}17$ tons per sq. in.

These values are less than the permissible shear and tensile stresses for bolts, and the bolt size will be satisfactory.

The gussets must be designed to take the upward moment produced by the upthrust from the foundation. The pressure per square foot on the foundation produced by the axial load and by the bending moment is shown in Fig. 28, and the gussets must withstand the upthrust from the portion of the base plate outside the stanchion shaft, *i.e.*, on the cantilever arm of $11\frac{1}{4}''$.

Total upthrust due to axial load $= \frac{4{\cdot}3 \times 11\frac{1}{4} \times 20}{12 \times 12} = 6{\cdot}7$ tons.

Moment of this upthrust $= 6{\cdot}7 \times 5\frac{5}{8} = 37{\cdot}7$ tons ins.

Total upthrust due to bending moment $= \frac{15{\cdot}05 \times 11\frac{1}{4} \times 20}{12 \times 12}$
$= 23{\cdot}5$ tons.

Moment of this upthrust $= 23{\cdot}5 \times 6\frac{1}{4} = 146{\cdot}9$ tons ins.

Total B.M. taken by gussets $= 37{\cdot}7 + 146{\cdot}9 = 184{\cdot}6$ tons ins.

The bending moment will be shared between the outer gussets and the intermediate gusset.

Proportion taken by each outer gusset $= \frac{184{\cdot}6 \times 7}{20} = 64{\cdot}6$ tons ins.

Proportion taken by intermediate gusset $= \frac{184{\cdot}6 \times 6}{20} = 55{\cdot}4$ tons ins.

The outer gussets are $16'' \times \frac{1}{2}$, and the inner gusset $16'' \times \frac{3}{8}$.

Z of outer gusset $= BD^2/6 = \frac{\frac{1}{2} \times 16 \times 16}{6} = 21{\cdot}3$

Z of inner gusset $= \frac{\frac{3}{8} \times 16 \times 16}{6} = 16.$

Stress in outer gusset $= \frac{\text{B.M.}}{Z} = \frac{64{\cdot}6}{21{\cdot}3} = 3$ tons per sq. in.

Stress in inner gusset $= 55{\cdot}4/16 = 3{\cdot}5$ tons per sq. in.

In view of the fact that the sloping edges of the gussets are in compression and unstiffened, these stresses are quite high enough for good workmanship. Strictly speaking, the sloping edges should be stiffened by a small flat, say $2\frac{1}{2}'' \times \frac{3}{8}''$, connected to the gusset with intermittent fillet welds.

The welds connecting the stanchion shaft to the gussets must resist the bending moment and the axial load.

Force in welds due to B.M. $= \dfrac{\text{B.M.}}{\text{flange centres}} = \dfrac{800}{18\frac{3}{4}}$

$= 42{\cdot}7$ tons.

Force in welds due to axial load $= 25/2 = 12{\cdot}5$ tons.

Total force in welds $= 42{\cdot}7 + 12{\cdot}5 = 55{\cdot}2$ tons.

Proportion of force carried by outer gusset $= \dfrac{55{\cdot}2 \times 7}{20}$

$= 19{\cdot}3$ tons.

Proportion of force carried by inner gusset $= \dfrac{55{\cdot}2 \times 6}{20}$

$= 16{\cdot}6$ tons.

Using $\frac{1}{4}''$ side fillets at 0·88 ton per lineal inch.

Length of weld required in outer gusset $= 19{\cdot}3/0{\cdot}88 = 22''$

Length of weld required in inner gusset $= 16{\cdot}6/0{\cdot}88 = 19''$

On the outer gussets, the weld will extend the full depth of 16″ for the fillet outside the shaft, and for a length of say 8″ for the inside fillet, 8″ being approximately as far as the weld can be laid efficiently in such a position.

The welds on the inner gusset must be checked against the effect of uplift produced by the holding down bolts on the windward side of the base, the pull on the gusset being equal to the tension in one bolt.

Tension per bolt $= 4{\cdot}86$ tons.

Moment of this tension $= 4{\cdot}86 \times 8\frac{1}{4} = 40{\cdot}1$ tons ins.

Bending stress $F_b = \text{B.M.}/Z = \dfrac{40{\cdot}1 \times 6}{2 \times 16 \times 16} = 0{\cdot}47$ tons per in.

Shear stress $F_s = \dfrac{4{\cdot}86}{2 \times 16} = 0{\cdot}15$ tons per in.

Resultant force per inch of weld $= \sqrt{0{\cdot}47^2 + 0{\cdot}15^2} = 0{\cdot}49$ ton.

$\frac{3}{16}''$ fillets are sufficient for strength, but for a $\frac{1}{2}''$ plate, the minimum size of weld desirable is $\frac{1}{4}''$ and a $\frac{1}{4}''$ fillet will be used.

The outer gussets will be connected to the base plate with a $\frac{1}{4}''$ fillet.

The holding-down bolts are shown carried up above the base plate and tightened down on washer plates resting on $3'' + \frac{1}{2}''$ flat stiffeners running between the gussets and connected to the gussets with $\frac{1}{4}''$ fillets.

The thickness of the base plate is fixed up to resist the upthrust on the portion projecting outside the main gusset. This upthrust will be a maximum on the extreme leeward edge of the base, as shown in Fig. 28, and at this point amounts to 23·9 tons per square

foot, or 372 pounds per square inch. This pressure decreases to zero near the centre of the base, and as the bulk of the pressure is due to wind load, it is usually considered safe to design to an average pressure of 75% of 372 pounds, *i.e.*, 279 lbs. (say 280 lbs.) per square inch.

Upthrust per inch run on $3\frac{1}{2}$" cantilever $= 280 \times 3\frac{1}{2} = 980$ lbs.

Moment of this upthrust $= 980 \times 1\frac{3}{4} = 1715$ lbs. ins.

Assuming a permissible stress of 10 tons, or 22,400 lbs. per square inch.

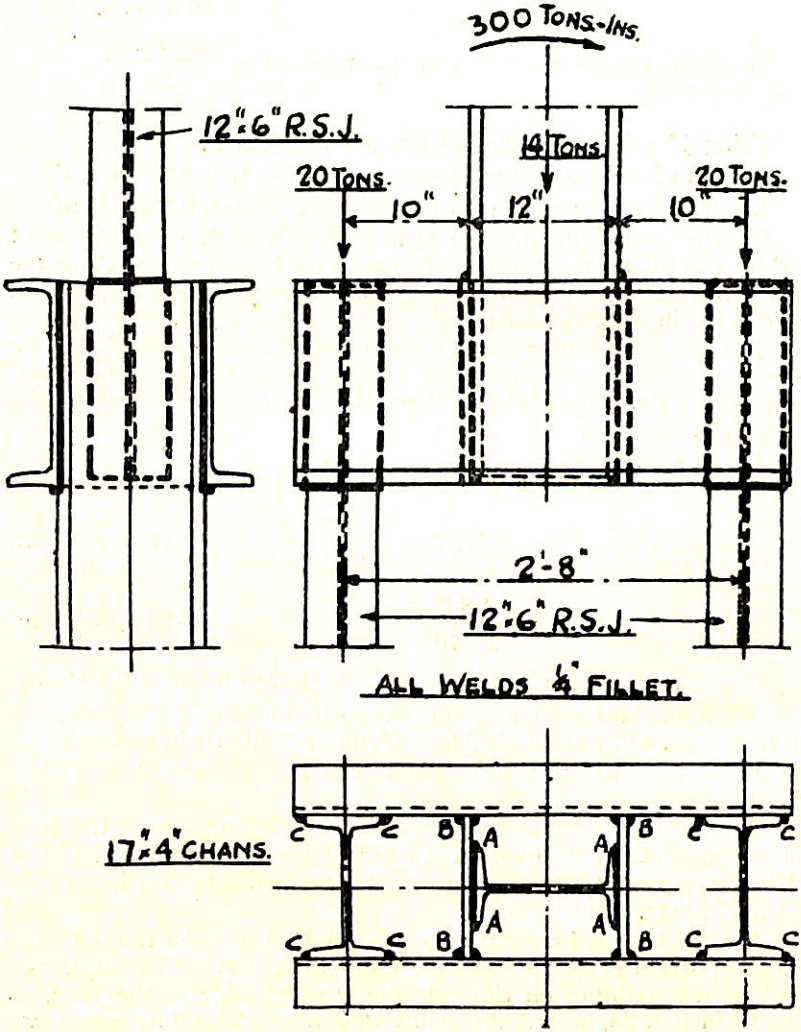

Fig. 30.—Stanchion Crane Cap, subject to Bending Moment.

Z required in plate = 1715/22,400.
Z given = $B t^2/6$

$$\therefore \quad t = \sqrt{\frac{1715 \times 6}{22{,}400}} = 0{\cdot}68'' \text{ (say } \tfrac{3}{4}'' \text{ plate).}$$

The welds connecting the web of the stanchion shaft to the flange are designed to take the shear, which consists of the side wind pressure plus the surge effect from the overhead crane, where the stanchions support a crane gantry. The shear per lineal inch equals the total shear divided by the depth of the stanchion web, 18", and is usually very small in stanchions of this type. To avoid corrosion, these welds are usually made continuous, and a $\frac{3}{16}''$ fillet will be ample.

Stanchion Crane Cap subject to Bending Moment.

Fig. 30 shows the layout of a typical welded crane cap. It is necessary to check the strength of welds A, B and C, and also of the 17" × 4" channels.

Welds A.—The roof leg is welded on both sides, and also top and bottom to the diaphragm plate. For simplicity of calculation, a throat thickness of 1" will be used as a basis.

I_{xx} of side fillets = $4\,BD^3/12 = \dfrac{4 \times 1 \times 16 \times 16 \times 16}{12} = 1366$

I_{xx} of top and bottom fillets (approx.) = $4\,Ar^2 = 4 \times 6 \times 8 \times 8$
$= 1536$

Total I_{xx} = 1366 + 1536 = 2902

$Z = I/y = 2902/8 = 362{\cdot}8.$

F_b = B.M./Z = 300/362·8 = 0·83 tons per square inch.

Total length of weld = 4 (16 + 6) = 88"

Shear stress F_s = 14/88 = 0·16 tons per square inch.

Resultant stress $F_r = \sqrt{F_b^2 + F_s^2} = \sqrt{0{\cdot}83^2 + 0{\cdot}16^2}$
$= 0{\cdot}85$ ton per square inch.

$\frac{1}{4}''$ fillets will take 0·88 tons per inch run, and will be used.

Welds B.

Load per weld due to B.M. = $\dfrac{300}{12{\cdot}5 \times 8} = 3$ tons.

Load per weld due to vertical load = 14/8 = 1·75 tons.
Total load per weld = 3 + 1·75 = 4·75 tons.
Load per inch of weld = 4·75/16 = 0·3 ton
(Assuming 16" effective depth).

It is not desirable to use welds of less than $\frac{1}{4}''$, and this size will be adopted.

Welds C.—These must take the bending moment plus the roof load plus the load from the crane reaction. As the crane load is a rolling load, it is usual to assume that 75% of this load may be acting on each side of the crane leg in turn.

$$\text{Total load on each run of weld} = \frac{300}{4 \times 32} + \frac{14}{8} + (75\% \text{ of } 20/2)$$

$$= 2{\cdot}3 + 1{\cdot}8 + 7{\cdot}5 = 11{\cdot}6 \text{ tons.}$$

Load per inch of weld = 11·6/16 = 0·73 ton.

$\frac{1}{4}$" fillets will be sufficient.

17" × 4" Channels.

$$F_b = \text{B.M./Z} = \frac{300}{2 \times 61{\cdot}2} = 2{\cdot}3 \text{ tons per square inch.}$$

These are, therefore, of ample strength.

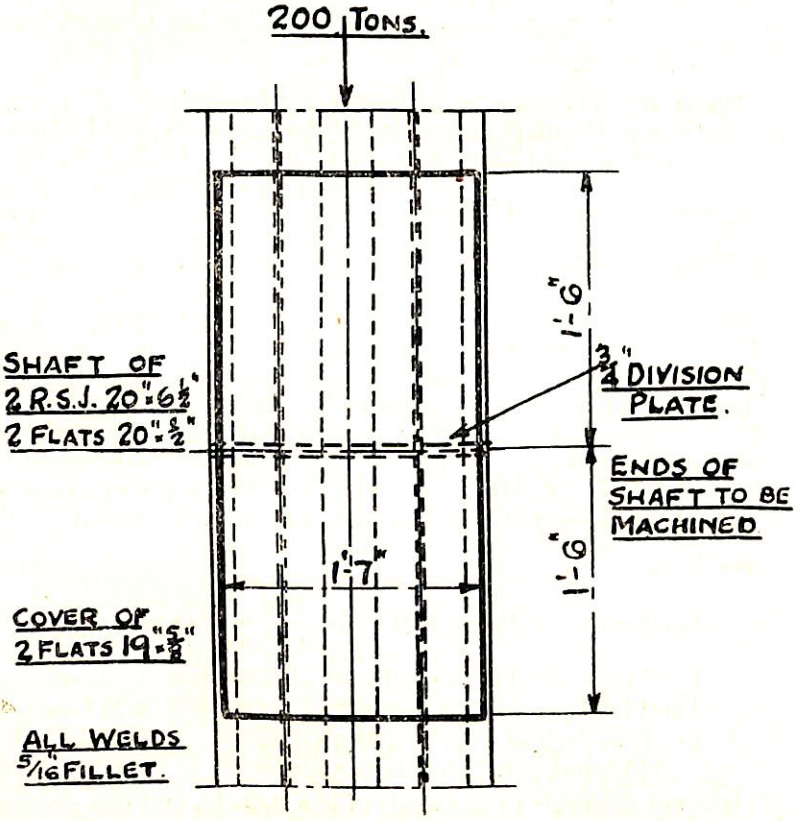

Fig. 31.—Stanchion Splice.

Stanchion Splices.

In multi-storey buildings, splices are usually located immediately above a floor level, and it is therefore not exceptionally difficult to weld these connections. Fig. 31 shows a typical detail incorporating both side and end fillets. It is not desirable to have side fillets only, even if they are strong enough for the load, as there would be a large concentration of stress in the side welds and very little stress at the centre line of the covers. The side and end fillets give also a more "natural" detail, with the cover serving as a boss on the stanchion shaft. This is another example of the analogy of a welded detail to a casting.

The end fillets are in tension and will therefore be assumed to have the same strength per inch as the side fillets. It may also be assumed that 50% of the load is transferred by bearing, for which purpose the butting ends of the shift must be machined.

Total load on splice = 200 tons.
Load taken by covers = 100 tons.
Load taken by each cover = 50 tons.
Sectional area of one flange of stanchion
$= (2 \times 6\frac{1}{2} \times 0{\cdot}82) + (20 \times \frac{1}{2}) = 20{\cdot}66$ sq. inches.

Sectional area required in cover $= 20{\cdot}66/2 = 10{\cdot}33$ sq. inches.

The cover is 19″ wide to allow for side fillets, and will be made $\frac{5}{8}$″ thick to give the required area.

Using $\frac{5}{16}$″ fillets with a value of 1·11 ton per inch, minimum total length of weld required on each side of joint $= 50/1{\cdot}11 = 45$″.

It will be noticed that on a double joist stanchion of this type, it is not possible to introduce web splices on both sides of each joist. A splice plate can be included on the outside of each web, to assist in keeping the stanchion in proper alignment. Alternatively, web splices can be omitted, and a bearing plate introduced, as shown in Fig. 31. If necessary, web cleats can also be added.

Beams and Plate Girders.

Basic Beam formula :—

$$f/y = M/I = E/R$$

Preliminary calculations for web and flange areas may be based on the assumption that the web takes all the shear, and the flanges take the bending moment.

In Fig. 32—Let A_F = flange area.
F_F = flange force.
S_V = vertical shear at any point.
S_H = horizontal shear at any point.

Then assuming the stress in the flange is approximately constant over all the flange cross section, and is equal to the extreme fibre stress f.

$$F_F = A_F \times f$$
$$M.R. = F_F \times d = A_F \times f \times d$$
$$M = M.R.$$

$$\text{and } A_F = F_F/f = \frac{M}{f \times d}$$

Preliminary calculations for the flange area may be made on this basis, and the section finally checked by the basic beam formula,

$$f = M\,y/I$$

The welds connecting the flanges to the web take the horizontal shear S_H

$$S_H = \frac{S_V \times G}{I} = \frac{S_V \times A_F \times (d/2)}{I}$$

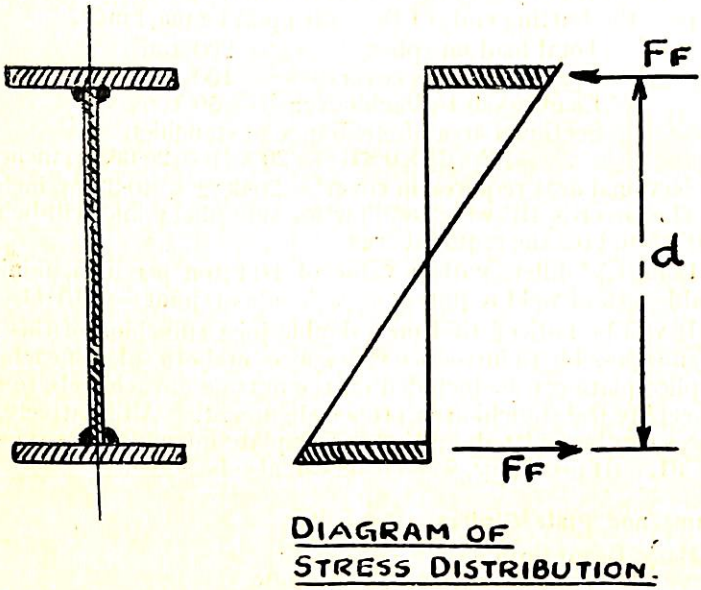

Fig. 32.

This formula, which is the standard formula for finding the horizontal shear in girder flanges, gives the shear per lineal inch of weld in each flange, and the fillet size is fixed up to suit. The fillets to the compression flange should always be made continuous, but those on the tension flange may be intermittent if the load permits, and there is no likelihood of corrosion between the flange and the web.

For girders taking heavy loads, the flange plates will be thicker at the centre than at the ends of the girder. It is not necessary,

however, to have separate plates as in riveted work. The centre portion of the flange can be made in one thickness (if not too great) and the joint between this plate and the end plates made as shown in Fig. 8, where the weld develops the strength of the smaller plate only. If for any reason it should be found necessary to pile additional plates outside the first plate, the outer plates should be narrower than the inner plate to allow for downhand welding and the longitudinal welds connecting the outer plate must take the horizontal shear between the plates, the shear being calculated from the formula given above, using the area of the additional plate only as the value of A_F.

Plate Girder Stiffeners.

The loads in the stiffeners are found in exactly the same way as for riveted construction, and the welds connecting the stiffeners to the web must be sufficient to transmit the load taken by the stiffener. Stiffeners for welded work consist usually of flats and the welds may be intermittent if of sufficient strength. As the stiffeners act as struts, it is undesirable to have too large a gap between the welds, 6″ being a fairly usual maximum. The inner corners of the stiffeners are bevel cut to clear the flange to web welding. Fig. 33 shows a typical arrangement of welds to plate girder stiffeners.

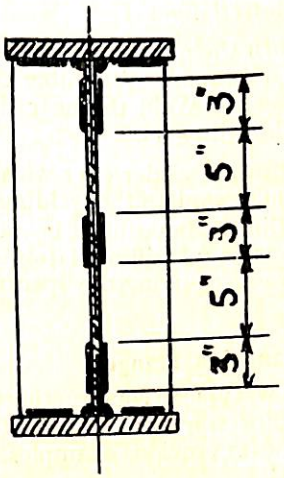

Fig. 33.—Typical Arrangement of Stiffeners.

Generally, it will be seen that so far as plate girders are concerned, welded construction shews considerable economy as compared with riveted work. The construction is much simpler, angle sections

being eliminated completely, while the welding operations are comparatively straightforward, the jigs being so constructed that the girder can be rotated for all welds to be laid in a downhand position. The economy is particularly marked where a large number of similar girders are required, as the cost of jigs per girder is fairly low. For small quantities, if special jigs have to be made, the cost of these may cancel out other economies, but it is not a difficult matter to design adjustable jigs to serve girders of slightly different sizes. The jig designer should, in fact, always bear in mind that the jigs for one job may be required at a later date to serve a different size of girder.

Procedure for Welding Plate Girders.

It is a fixed rule for welding practice that as far as possible all welding should be done in the downhand position. This is the natural position which enables the operator to work in reasonable comfort, and therefore to ensure good work. Vertical welding and overhead welding are always difficult and hazardous, and should never be attempted if downhand welding can be arranged. On this principle the operation for welding a plate girder will be as follows :

1st Operation.—Lay one flange on the welding table or stallage, and assemble and weld all the web stiffeners to this flange.

2nd Operation.—Lower web plate into position between the stiffeners and weld web to flange.

3rd Operation.—Turn the girder over on to its side to bring the web into a horizontal position. Assemble the second flange and clamp it to the stiffeners. Weld the near side of the web to the second flange and to the stiffeners.

4th Operation.—Turn the girder over with the other side of the web now uppermost and complete the welding.

The procedure outlined above may be taken as a general instruction for welded work, and will obviate many of the difficulties met with in welding practice, giving the operator every opportunity to produce good work.

Crane Brackets on Stanchion Flanges.

Figs. 34 to 37 show typical constructions for the more usual types of brackets. For simplicity, calculations are based on a throat thickness of 1", as in previous examples.

Referring to Fig. 34.

I_{xx} of welds $= 2Ar^2 = 2 \times 6 \times 1 \times 6 \times 6 = 432$

Z of welds $= I/y = 432/6 = 72$

$$F_b = \text{B.M.}/Z = Pe/Z = \frac{12 \times 6}{72} = 1 \text{ ton per lineal inch.}$$

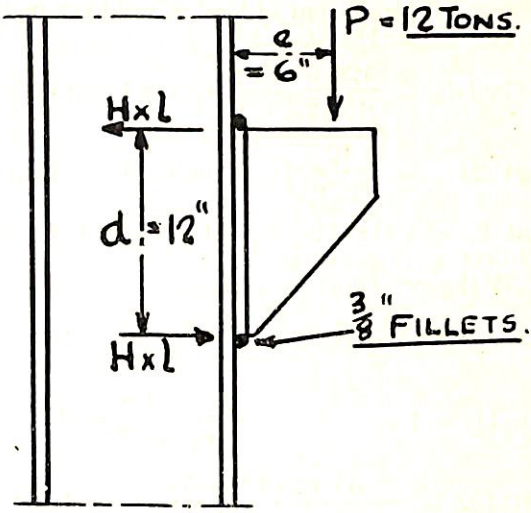

Fig. 34.—Crane Brackets with End Fillets.

$$F_s = P/A = \frac{12}{2 \times 6 \times 1} = 1 \text{ ton per lineal inch.}$$

$F_R = \sqrt{1^2+1^2} = 1{\cdot}41$ tons per lineal inch.

An alternative procedure is as follows :—

H = horizontal component of load in weld per inch.

l = length of weld = 6".

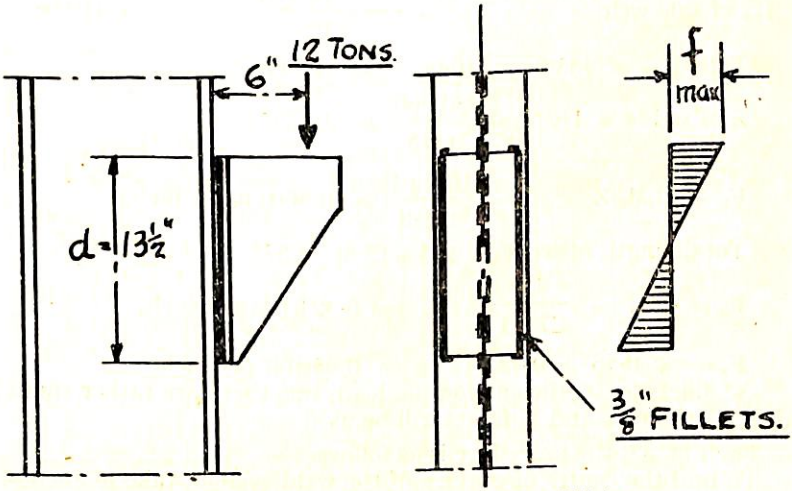

Fig. 35.—Crane Brackets with Side Fillets.

V = vertical component of load in weld per inch.

B.M. = $Pe = H \times l \times d$

$$\therefore \quad H = Pe/l\,d = \frac{12 \times 6}{6 \times 12} = 1 \text{ ton per lineal inch.}$$

$$V = P/2\,l = \frac{12}{2 \times 6} = 1 \text{ ton per lineal inch.}$$

Resultant $F_R = \sqrt{1^2 + 1^2} = 1{\cdot}41$ tons per lineal inch.

Use $\frac{3}{8}$" fillets.

For Fig. 35 the procedure is as follows :—

$$I_{xx} \text{ of welds} = 2\,BD^3/12 = \frac{2 \times 1 \times 13{\cdot}5^3}{12}$$

$$Z_{xx} \text{ of welds} = I/y = \frac{2 \times 1 \times 13{\cdot}5^3}{12 \times 6{\cdot}75}$$

$$F_b = \text{B.M.}/Z = \frac{6 \times 12 \times 12 \times 6{\cdot}75}{2 \times 1 \times 13{\cdot}5 \times 13{\cdot}5 \times 13{\cdot}5} = 1{\cdot}19 \text{ tons per inch.}$$

$$F_s = P/A = \frac{12}{2 \times 13{\cdot}5 \times 1} = 0{\cdot}44 \text{ ton per inch.}$$

$$F_R = \sqrt{1{\cdot}19^2 + 0{\cdot}44^2} = 1{\cdot}27 \text{ tons per inch.}$$

Use $\frac{3}{8}$" fillets.

For Fig. 36, the procedure is as follows :—

$$I_{xx} \text{ of top and bottom welds} = 2 \times 5 \times 1 \times 6{\cdot}75 \times 6{\cdot}75 = 455{\cdot}6$$

$$I_{xx} \text{ of side welds} = \frac{2 \times 1 \times 13{\cdot}5^3}{12} = 410{\cdot}06$$

Total $I_{xx} = 455{\cdot}6 + 410{\cdot}06 = 865{\cdot}66$

$$Z \text{ of welds} = I/y = \frac{865{\cdot}66}{6{\cdot}75} = 128{\cdot}3$$

$$F_b = \text{B.M.}/Z = \frac{6 \times 12}{128{\cdot}3} = 0{\cdot}56 \text{ ton per inch.}$$

Total length of weld = $2\,(5 + 13{\cdot}5) = 37$"

$$F_s = P/A = \frac{12}{37 \times 1} = 0{\cdot}324 \text{ ton per inch.}$$

$$F_R = \sqrt{0{\cdot}56^2 + 0{\cdot}324^2} = 0{\cdot}65 \text{ ton per inch.}$$

$\frac{3}{16}$" fillets are sufficient for the load, but these are rather small for a joist flange, and $\frac{1}{4}$" fillets will be used.

For Fig. 37, the procedure is as follows :—

To find the centre of gravity of the weld system, take moments about the top flange of the bracket.

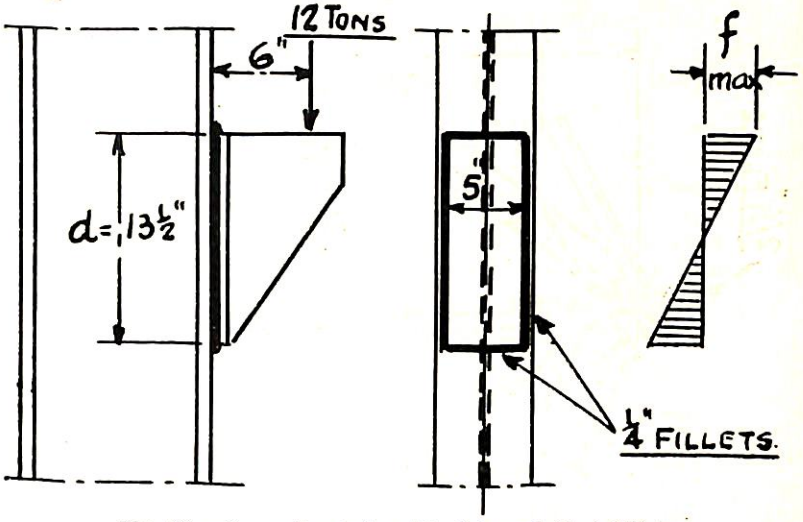

Fig. 36.—Crane Brackets with Side and End Fillets.

Moment of flange weld $= 6 \times 0 = 0$

Moment of web welds $= 2 \times 11 \times 7 = 154$

$$y_t = \frac{\text{total moment}}{\text{total area}} = 154/28 = 5\tfrac{1}{2}''$$

$$y_c = 12\tfrac{1}{2}'' - 5\tfrac{1}{2}'' = 7''$$

$$I_{xx} = (6 \times 5\tfrac{1}{2}^2) + \left(\frac{2 \times 11^3}{12}\right) + (2 \times 11 \times 1\tfrac{1}{2}^2)$$

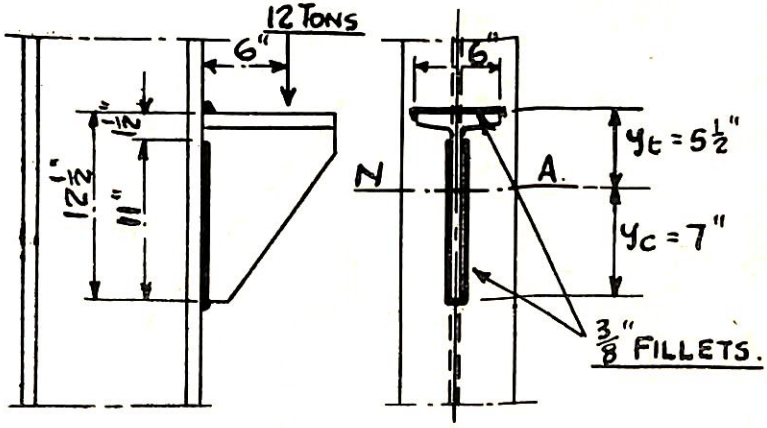

Fig. 37.—Unsymmetrical Crane Brackets.

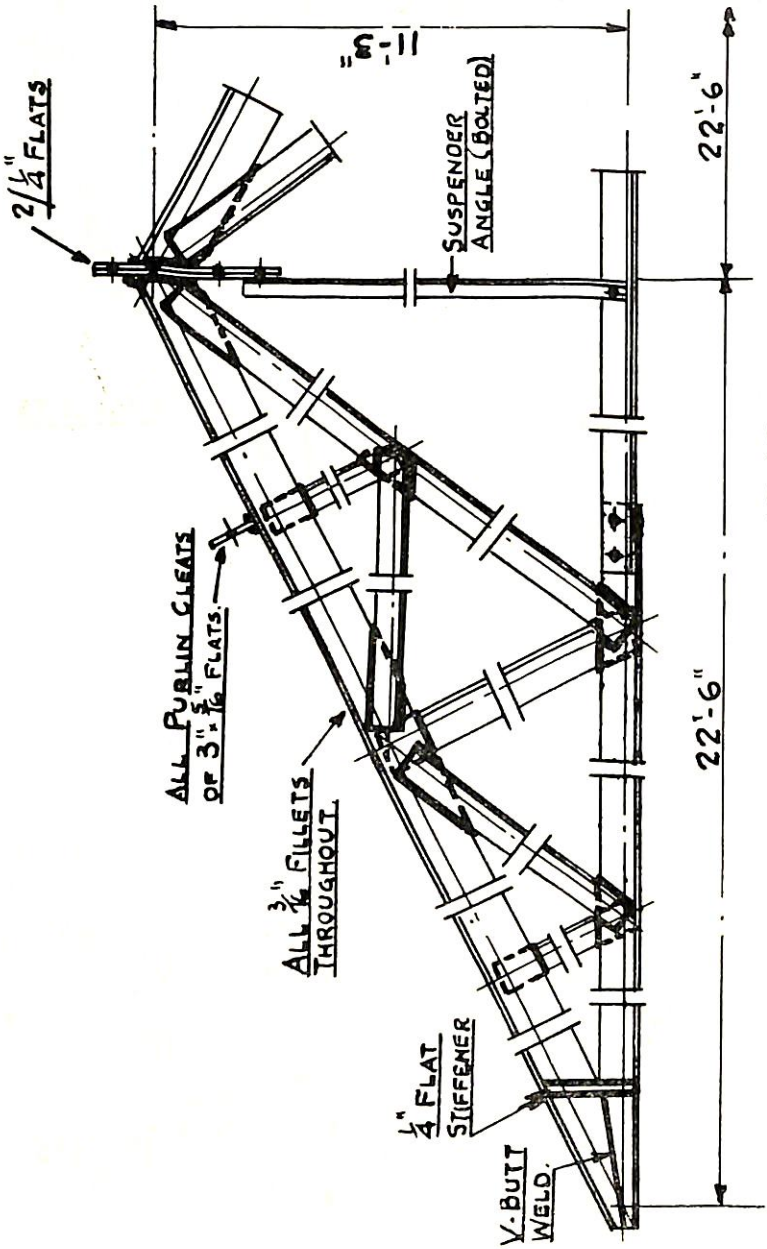

Fig. 38.—Typical Details of Roof Truss.

$= 181{\cdot}5 + 221{\cdot}8 + 49{\cdot}5 = 452{\cdot}8$

$Z_t = 452{\cdot}8/5\frac{1}{2} = 82{\cdot}3$

$Z_c = 452{\cdot}8/7 = 64{\cdot}7$

Maximum tensile stress $= B.M./Z_t = 72/82{\cdot}3$
$= 0{\cdot}87$ ton per inch.

Maximum compressive stress $= B.M./Z_c = 72/64{\cdot}7$
$= 1{\cdot}11$ tons per inch.

$F_s = P/A = 12/28 = 0{\cdot}43$ ton per inch.

Resultant tensile stress $= \sqrt{0{\cdot}87^2 + 0{\cdot}43^2} = 0{\cdot}97$ ton per inch.

Resultant compress. stress $= \sqrt{1{\cdot}11^2 + 0{\cdot}43^2} = 1{\cdot}19$ tons per inch.

$\frac{5}{16}$" fillets would do for the flange weld, but $\frac{3}{8}$" fillets are required for the web, and all welds will be made $\frac{3}{8}$" fillets.

Roof Truss Details.

The loads in the members of roof trusses are comparatively small, and generally there is no difficulty in arranging welds of adequate strength without having to introduce gusset plates. Attention is mainly needed in arranging the details of the joints to facilitate welding. In Fig. 38, the legs of the rafters and the main tie are in the same plane, with a V-butt weld at the shoe. The struts connect to the far side of the rafters and main ties, and the diagonals to the near side. By this procedure, it is fairly easy to arrange the weld runs and to fix up the sequence of operations.

For large trusses, the disposition and length of the welds is fixed up in accordance with the procedure given for Figs. 14 or 15, whichever is applicable, but for smaller trusses the length of weld provided is usually in excess of theoretical requirements, and the only consideration is to fix up a convenient detail.

The centre main tie and the centre suspender angle are despatched loose, and site-bolted to avoid site-welding. The remainder of the truss is shop-welded and despatched in two pieces. The detail at the apex is arranged to avoid site welding, and is quite satisfactory. The elimination of gussets effects a considerable economy in material, while the absence of rivet holes in the tension members also enables smaller sections to be used than are required for riveted work. The struts may be also designed as fixed ended, and this again is an economy. The principal difficulty with welded trusses, and with all braced frames, is in clamping the members together in correct relative position, and this to some extent offsets other economies. Allowing for this cost, welded trusses are still cheaper than riveted trusses, principally on account of the economy in material.

It will be noted in Fig. 38 that all members have welds on both sides of the truss. The sequence of operations is based on this principle, and the procedure will be as follows :—

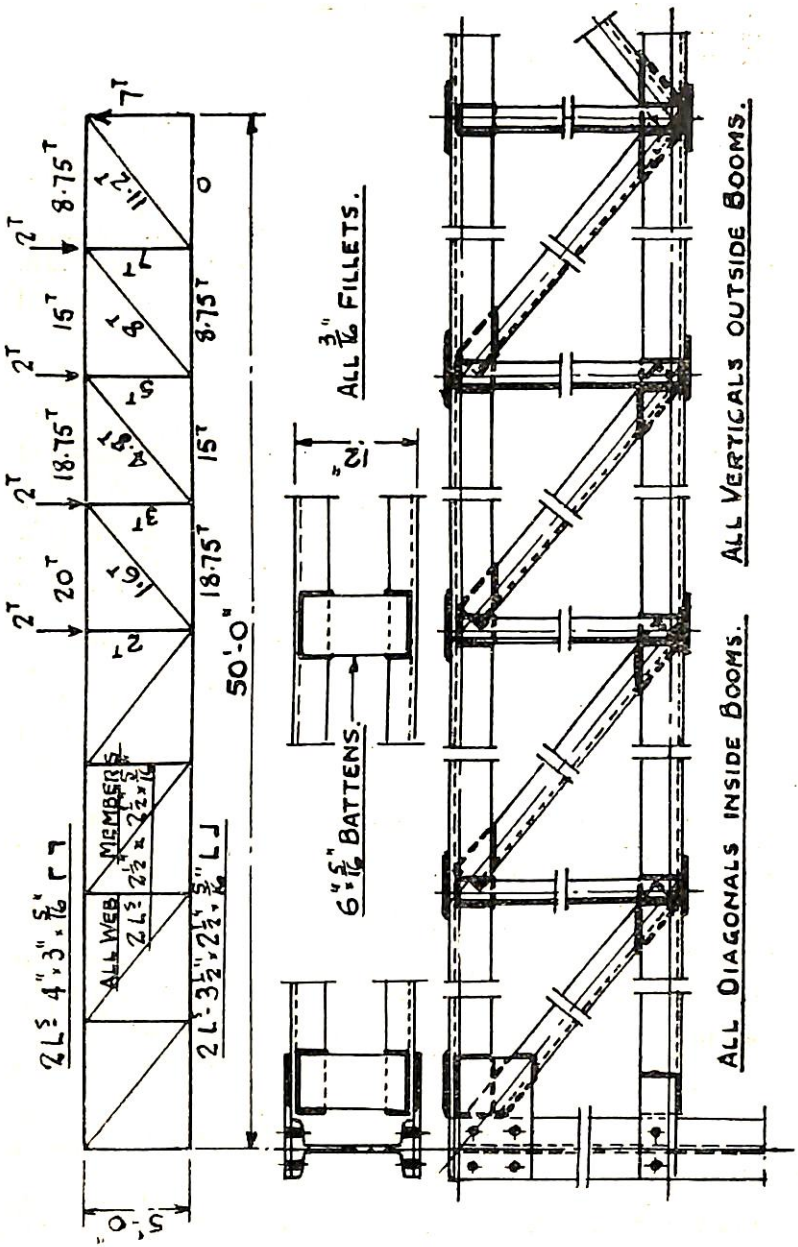

Fig. 39.—Typical Details of Lattice Girder.

1st Operation.—For each half of the truss, clamp all members in correct relative position, the truss lying flat on the assembly stallage.

2nd Operation.—Lay all welds on near side of truss, all members being thus tacked in position.

3rd Operation.—Turn truss completely over, and lay all welds on the other side.

It will be seen again that downhand welding is employed exclusively, and once the members are securely clamped, no great difficulty is likely to be met with in welding.

Lattice Girder Details.

Fig. 39 shows a typical layout for a 50-ft. span lattice girder. The booms have been opened out, partly to give rigidity to the girder, and also to allow the diagonals to be welded to the inside. The verticals have been placed outside the booms, and this arrangement makes it possible to get in the necessary lengths of weld runs without difficulty.

As in the case of the roof truss details, the lengths of the welds are ample for the loads.

If considered desirable to keep all the members in the same plane as nearly as possible, the legs of the diagonals can be turned outwards instead of inwards, with the outstanding leg notched at each end for welding inside the booms, but as the diagonals are tension members, this is not a vital point on small girders, where the sections and welds are of ample strength.

The weld arrangement at each joint follows the same general principle as for the roof truss. Each member has a near side and a far side weld at each end. Each half of the girder will be laid out and the members clamped together. The near side welds will then be laid, the half girder turned completely over, and the welds to the other side laid. The two halves of the girder are then brought together and the batten plates welded on, downhand welding being used for all operations.

Staircase Details.

Fig. 40 shows typical details of a welded staircase stringer, and is self-explanatory. It will be seen that compared with a riveted construction, the details are much simpler.

The stair treads are supported by flats acting as lugs, and the channel cutting at the top gives a much simpler detail than a riveted connection which is often complicated and costly.

Standardising of Welding Technique.

It is not possible to lay down hard-and-fast rules governing

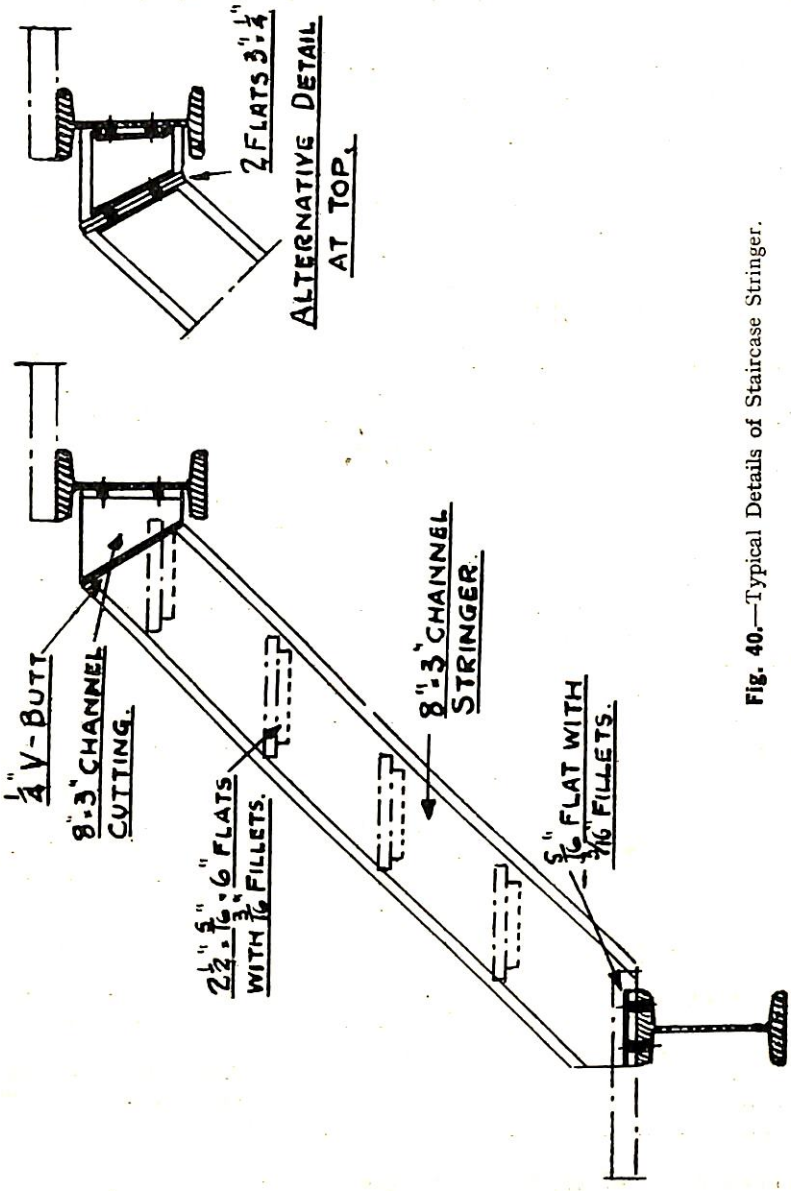

Fig. 40.—Typical Details of Staircase Stringer.

procedure, as most firms have their own standards, all of which will vary to some extent. There is also a considerable variety of electrodes. Moreover, in welded work more even than in riveted work, plant layout has a considerable effect on performance, while of equal importance is the necessity for making provision for the welder to operate continuously without undue fatigue. All of these points must be borne in mind in deciding procedure and in assessing electrode requirements.

A certain amount of general guidance can be given, and the notes following are intended to provide an approximate basis for procedure and calculation.

It can be stated that it is economical to deposit a given size of weld with the largest electrode the work will stand. For downhand welding, assuming no special difficulty is likely to be met with, electrodes should be 12 or 10 gauge for plates under $\frac{3}{16}$" thick, 8 gauge for plates $\frac{3}{16}$" to $\frac{5}{16}$" thick, 6 gauge for plates $\frac{5}{16}$" to $\frac{7}{16}$" thick, and 4 gauge for plates $\frac{1}{2}$" thick and over. Heavier electrodes $\frac{5}{16}$" and $\frac{3}{8}$" diameter are sometimes used for thicker plates.

For vertical, inclined and overhead welding, these sizes will vary slightly, and it is desirable to refer to the shops before specifying the rod gauges in these cases.

Electrodes are normally supplied in standard lengths of 18" and the approximate lengths of run which should be obtained from standard electrodes for fillet welds and for single V-butt welds are shown in Tables 2 and 3. These tables are for normal, straightforward work, and can only be approximately correct, as values will vary according to the fit-up of the work and according to the skill of the welder. Allowance has been made in the tables for normal wastage.

The amount of wastage is a somewhat indeterminate percentage. On heavy electrodes it will be less than on light electrodes. There will also be less waste for long weld runs than for short runs. The wastage also varies with individual operators. For 10 gauge electrodes the wastage will be from 15% to 20%, with a further 5% for short runs. For 4 gauge electrodes, the wastage will be from 10% to 15%, with a further 5% for short runs. In actual practice it is very rare that the estimated total feet run of electrode is found to be correct.

Tables 2 and 3 are based on actual practice of average structural work, with welders of average skill working under fairly satisfactory conditions.

Weld runs are specified as 10/12, 8/10, 6/9, etc., the first number indicating the gauge of the electrode, and the second number the length of the run in inches which is required to be obtained from a standard 18" electrode.

A convenient system of symbols for specifying welds is shown in Fig. 41, although most firms usually develop their own system.

TABLE 2.

Approximate Lengths of Run obtained from Standard 18″ Electrode for Fillet Welds.

Fillet Size.	Throat Thickness.	Area Sq. Ins.	Gauge of Electrode.	Length of Run.
$\frac{1}{8}$″	·088″	·012	12	10″
$\frac{1}{8}$″	·088″	·012	10	14″
$\frac{3}{16}$″	·133″	·023	10	8″
$\frac{3}{16}$″	·133″	·023	8	13″
$\frac{1}{4}$″	·176″	·039	8	8″
$\frac{1}{4}$″	·176″	·039	6	12″
$\frac{5}{16}$″	·221″	·058	6	8″
$\frac{5}{16}$″	·221″	·058	4	12″
$\frac{3}{8}$″	·265″	·081	4	8″

TABLE 3.

Approximate Lengths of Run obtained from Standard 18″ Electrode for Single V-Butt Welds.

Plate Thickness.	Area Sq. Ins.	Gauge of Electrode.	Length of Run.	Angle of Weld.
$\frac{1}{8}$″	·019	12	14″	70°
$\frac{1}{8}$″	·019	10	18″	70°
$\frac{3}{16}$″	·037	10	9″	70°
$\frac{3}{16}$″	·037	8	14″	70°
$\frac{1}{4}$″	·060	8	7″	70°
$\frac{1}{4}$″	·060	6	10″	70°
$\frac{5}{16}$″	·088	6	7″	70°
$\frac{5}{16}$″	·088	4	9″	70°

SEALING RUN—One 10/12″ in each case—in Addition.

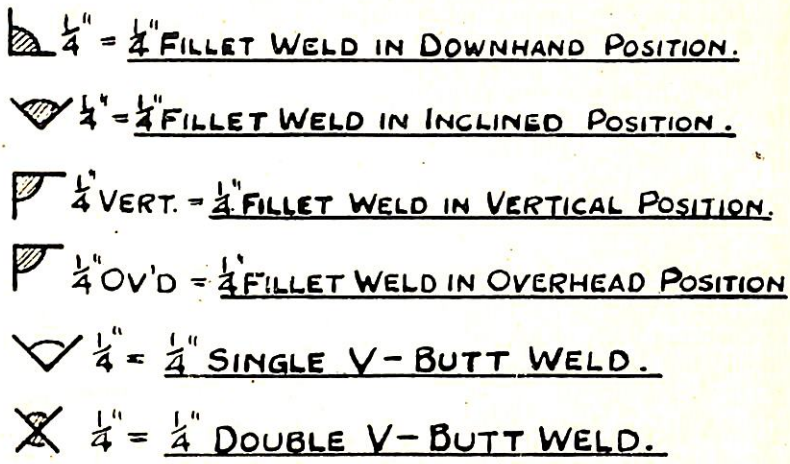
1/4" = 1/4" FILLET WELD IN DOWNHAND POSITION.
1/4" = 1/4" FILLET WELD IN INCLINED POSITION.
1/4" VERT. = 1/4" FILLET WELD IN VERTICAL POSITION.
1/4" OV'D = 1/4" FILLET WELD IN OVERHEAD POSITION
1/4" = 1/4" SINGLE V-BUTT WELD.
1/4" = 1/4" DOUBLE V-BUTT WELD.

Fig. 41.—Typical Weld Symbols.

Multiple Run Welds.

For weld sizes larger than $\frac{1}{4}$", it is better practice, and probably more economical to lay these in two or more runs. This point is disputed so far as fillet welds are concerned, but there can be little doubt with regard to butt welds.

Generally 2-run welds are undesirable, as they necessitate weaving of the second run to obtain the required profile. If more than one run is necessary, it is better to have three runs rather than two.

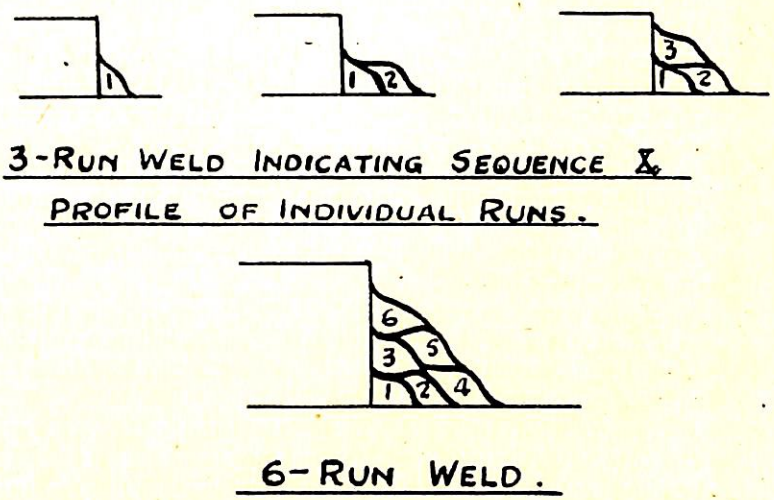

Fig. 42.—Multiple Run Welds.

It is also better practice in multiple runs to use the same gauge of electrode for all runs, although some firms do not observe this rule.

The general make-up of multiple-run welds is shown in Fig. 42. It will be seen that the profile of each run is so rounded off as to prevent the slag from the coating of the electrode percolating into the weld. This is a most important point which requires strict watching, and each run should be inspected and passed before the next run is laid.

EXERCISES.

These exercises are intended to give the reader an opportunity of checking up his knowledge of the essential design sections, but should not be attempted until the subject matter has been fully grasped.

1. A $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$ angle stressed to 8 tons per square inch with the full area effective, is to be connected to a gusset plate. Sketch a suitable detail, giving necessary calculations and weld sizes.
2. A $12'' \times \frac{1}{2}''$ flat tension member in a bridge truss is to be connected to a gusset plate, and is stressed to 7 tons per square inch. Sketch a suitable connection giving necessary calculations and weld sizes.
3. A $14'' \times 6''$ R.S.J. floor beam has to be connected into the web of an $18'' \times 6''$ main girder, the top flanges being flush. The reaction is 15 tons, and the site connection is to be bolted. Sketch a suitable connection, giving necessary calculations and weld sizes.
4. An $18'' \times 6''$ R.S.J., with an end reaction of 16 tons, is to be connected to the flange of a $12'' \times 8''$ R.S.J. stanchion, with a connection similar to that shown in Fig. 17, the beam being 5″ eccentric. Sketch a suitable detail, giving all necessary calculations.
5. A $14'' \times 6''$ R.S.J. is to be connected to each flange of a $12'' \times 6''$ R.S.J. stanchion, with a connection similar to that shown in Fig. 23. Prepare a sketch of a suitable detail giving necessary calculations. Assume a maximum working stress of 8 tons per square inch.
6. A column consisting of $10'' \times \frac{5}{8}''$ flange plates and an $18'' \times \frac{3}{8}''$ web supports a vertical load of 100 tons. Design a suitable welded base, assuming a safe bearing pressure of 20 tons per square foot on the concrete foundation.
7. A column with $10'' \times \frac{5}{8}''$ flange plates and a $20'' \times \frac{3}{8}''$ web supports a vertical load of 20 tons, and also has to resist a bending moment at the base of 500 tons inches, due to wind

pressure and crane surge effect. The allowable bearing pressure on the concrete foundation is 30 tons per square foot. Design a suitable welded base, giving all necessary calculations.

8. A valley stanchion, consisting of two crane legs of 14″ × 6″ R.S.J. section, spaced at 33″ centres, and a roof leg of 12″ × 6″ R.S.J. section, all arranged generally, as shown in Fig. 30, supports a vertical roof load of 10 tons, and a bending moment, due to roof wind, of 200 tons inches. Each crane leg also supports a vertical load of 24 tons. Prepare a sketch of a suitable crane cap, giving necessary calculations.

9. A plate girder 60-ft. span, supports an equally distributed load of one ton per lineal foot, and in addition eleven concentrated loads, each of 6 tons, equally spaced at intervals of 5-ft. Find a suitable central section for the girder, assuming an overall depth of not more than 5-ft., fix up the size of welds connecting the web to the flanges, sketch in the positions of the stiffeners, assuming the concentrated loads are the reactions from 10″ × 4½″ floor beams, with top flanges flush with the top of the girder, and give typical details of the welding of stiffeners to web and flange.

10. A lattice girder of 50-ft. span, supports three vertical loads, each of six tons, occurring at quarter points and at the centre. The overall depth of the girder is not to exceed 5-ft. Prepare a design for the girder by any known method, and show details of the welded joints.

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